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Journal of the
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SED RESEARCH REPORT NO. 12

ON — BIOLOGICAL OXIDATION OF ALKALINE WASTES

BY — The Sanitary Engineering Research Committee, Industrial Wastes Section

From Research Data of — Collected Sources

Acknowledgment — The Sanitary Engineering Division gratefully recognizes the generosity and professional courtesy of the following persons in making these research data available to the Society for review, presentation and comment by the Industrial Waste Section of the Research Committee.

Walter Gardner	Gastonia, N.C.
A. W. Petty	Virginia Beach, Va.
J. O. Lindsay	Lyman, S.C.
F. J. Doutt	Canton, N.C.
E. J. Struzeski	Raleigh, N.C.

Synopsis

Biological action does occur at pH values of 9 to 11. Less oxidation results at the high pH values, but the capital and operating costs involved with neutralization are eliminated. Results include experiments with paper mill, textile, synthetic, domestic and mixed wastes.

Biological Oxidation of Alkaline Wastes

Introduction

It has become customary practice among sanitary engineers to discourage industry from the biological treatment of unneutralized alkaline wastes. Engineers have generally agreed that biological activity practically ceases at

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pH values above 10. This accepted assumption means that highly alkaline wastes must be neutralized before biological treatment. Neutralization costs money and often causes many operating difficulties. The overall result is that industry hesitates to treat highly alkaline organic wastes. It is the objective of this report to refute the above opinions. Research, Pilot Plant, and operating data are presented to support the thesis that biological treatment is practical and effective at high pH values.

Examples of Alkaline Biological Treatment

1. Champion Fibre and Paper Company Waste

The pulping waste from the Canton, North Carolina[1] plant was treated by an activated sludge process in a pilot plant from March to May 1955. Nitrogen and phosphorus were added to the waste and a high solids concentration was maintained in the aeration basin. The results are expressed in Table 1.

TABLE 1

*High Solids, Alkaline Aeration of Pulping Wastes

No. of Days Sampled	ph Max.	ph Avg.	ph Min.	%	BOD Reduction #/1000 ft.^{3}/day
41	11.0	9.8	7.3	80	74.9

* (3-4 hours detention time.)

2. Gastonia, N.C. Sewage and Waste Treatment

The City of Gastonia, N.C. treats sewage from about 35,000 people and industrial waste from several textile mills and other factories and commercial industries. The disposal plant is of the activated sludge type except that no primary sedimentation is provided. The aeration basins are loaded at the rate of 48.3# BOD/1000 ft.^{3}/day of aerator volume.[2] The pH of the sewage waste mixture varied widely and without any pattern as shown in Table 2.

TABLE 2

Variation in pH of Sewage-Waste Mixture at Present Gastonia, N.C. Sewage Treatment Plant

	Percent of time pH is above			
	6.0	7.5	8.5	9.5
12 noon—6 A.M.	100	45	28	11
6 A.M.—12 mid	100	51	40	21
12 mid—6 P.M.	100	42	30	17
6 P.M.—12 noon	100	39	26	19
Overall	100	45	32	17

Despite the fact that 17% of the time the pH was above 9.5 the BOD reduction at this plant averages well above 70% and often reaches 90%.

3. Research Results—Sawyer, Frame and Wold[3]

The researchers used five constant temperature baths with four 2.5 liter capacity aeration tubes for their activated sludge studies. Suspended solids of 1500 ppm were maintained in the tubes. All sludge used was developed from the cotton kiering waste mixtures using an initial seeding of sewage organisms.

In one experiment they found that, while pH of 10.5 was somewhat inhibitory to the mocro-organisms after 6 hours of aeration at 10°C and 20°C, no inhibition was observed at 30°C. Table 3 contains some of their major findings.

TABLE 3

Sawyer, Frame and Wold Studies on Effect of Alkalinity in the Aeration Mixture on Effluent BOD (Initial BOD 890 ppm)

Alkalinity (ppm)	Percent Reduction in B.O.D. After 5 Days at Initial pH 10.5	After 5 Days at Initial pH 11.0
180	93.6	
1650	93.0	
2090	94.4	
2410	92.7	
1780		90.3
2165		90.3
2490		91.8
3110		91.8

4. Virginia Beach, Virginia, Operating Data

This sewage plant process consists of pre-chlorination, screening, grit removal, primary sedimentation, chemical precipitation, filtration, final sedimentation, and post chlorination.[4] The pH of the raw sewage varies from 6.7-7.3. Lime is added to the sewage after settling. The filter influent pH varies from 9.6 in winter to above 10.0 in summer. Through the filter the pH drops to around 7.3-7.5. The 120 ft. diameter filter has been treating sewage with a pH of 9.0-10.0 for 10 years. The BOD reduction is in excess of 95%, the effluent is odorless and sparkling clear. Mr. Petty states "it seems to us unnecessary to spend time to determine if sewage can be treated on a trickling filter of 9.0 to 10.0 when here it is an established process, in fact the most efficient method of treatment."

5. Souther's Research Data on Biological Treatment of Highly Alkaline Textile Mill Waste-Sewage Mixture at Greensboro, N.C.

Laboratory pilot plant trickling filter experiments at Cone Mills Laboratory showed that biological activity was prevalent at an influent pH of 10.5.[5]

Table 4 summarizes the results at this pH when using a 2:1 recirculation rate over a one-month period. The BOD loading on the filter varied from 1.5-2.0 lbs./yd.^{3}/day.

TABLE 4

BOD Reduction of Alkaline Greensboro Sewage-Cone Mill Textile Waste Mixture by High Rate Trickling Filtration

	Initial Mixture	Settling Basin Effluent	(% reduction)
BOD (ppm)	471	198	58
pH	10.5	9.1	

6. Nemerow's Results on Pilot Plant Dispersed Growth Aeration of Alkaline Cotton Mill Finishing Waste.

The pilot plant consisted of a combination of holding basins, aeration tanks, and final settling basins.(6) The waste flow was constant at 2 gpm. The wastes originated from the kiering, bleaching and dyeing of cotton fabrics. This resulted in an alkaline, highly colored and pollutional waste. The pilot plant results are presented in Table 5.

*TABLE 5

Dispersed Growth Aeration of Alkaline Cotton Finishing Wastes of Pacific Mills, Lyman, S.C.

Raw Waste pH	Raw Waste 20°C, 5 day BOD (ppm)	Air Rate (ft.^{3}/#BOD)	Aeration Basin Effluent BOD (ppm)	Aeration Basin Effluent BOD (% red.)	pH	(°F)	Total Bacterial (Numbers) (per ml.)
11.4	1084	2210	580	46.4	8.3	66	61,000,000
11.3	951	1890	540	43.2	8.5	75	32,000,000
11.3	984	1520	654	33.6	8.2	86	88,700,000

* Represents 107-4 hour composit samples.

7. Laboratory Results at North Carolina State College on Dispersed Growth Aeration of Glucose Protein Wastes.

Unpublished laboratory research data(7) on the dispersed growth aeration of a mixture of 43.2% protein and 56.8 glucose are available. The data are summarized in Table 6. The pH was adjusted daily to the values indicated in the table.

Discussion

The evidence presented in this report clearly verifies that some biological oxidation occurs at pH values of 9 to 11. This conclusion is based on results

TABLE 6

N. C. State College Laboratory
*BOD Reductions at Various pH Values

pH	7.0	8.0	9.5	11.0
BOD (5) Reduction	66.1	56.3	59.6	23.2

* Averages of 6 consecutive days of aeration with batch feedings and pH adjustments daily.

obtained with pulp and paper mill wastes, textile wastes, domestic sewage and mixtures of each as well as synthetic wastes. The initial pH in some cases varied between 7 and 11 while in others it remained constant at 10-11. In at least one case actual total bacterial counts showed numbers over 10 million per ml. In several cases the pH dropped during biological treatment. As much as 90% BOD reduction was obtained by biological activity at pH values as high as 11.0. No neutralization was utilized in any of these studies.

It is evident that the efficiency drops in some cases as the pH rises above 9. Bacteria oxidizing the organic matter prefer a neutral or somewhat alkaline environment, but can tolerate considerable alkalinity and higher pH values than 9. It is not the purpose of this report to discuss the mechanisms and reactions involved at high pH values.

Engineering Implication

Designing engineers should consider from an economic standpoint the biological treatment of highly alkaline, organic wastes. Somewhat less efficiency results at very high pH values whereas greater BOD reductions occur at lowered pH values. Unfortunately, lowering the pH often results in high capital and daily operational costs. In all cases the capacity of the receiving stream to assimilate the pollutional load should govern the degree to treatment required. Economics, however, should control the means whereby the treatment is carried out.

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Credit

This research report, which is one of a series of professional contributions by the Committee on Sanitary Engineering Research,

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Paper 1270 SA 3

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SED RESEARCH REPORT NO. 13

ON — MILK AND FOOD PROBLEMS IN CIVIL DEFENSE

BY — The Sanitary Engineering Research Committee, Public Health Engineering Section.

From Research Data of — Milton J. Foter, Chief, Food Microbiology; Harold L. Jeter, Training Specialist, and Keith H. Lewis, Chief, Milk and Food Research, Robert A. Taft, Sanitary Engineering Center, Division of Sanitary Engineering Services, BSS, Public Health Service, U.S. Department of Health, Education, and Welfare.

Acknowledgment: The Sanitary Engineering Division gratefully recognizes the generosity and professional courtesy of the authors in making these research data available to the Society for review, presentation and comment by the Public Health Engineering Section of the Research Committee.

Synopsis

Milk and Food resources and water supplies are of vital concern in emergency conditions. They are subject to hazards caused by radioactivity and by sabotage. Problems of food safety and salvage are reviewed in this digest of the paper prepared for Public Health Civil Defense Training.

INTRODUCTION

In the event of a national emergency, with subsequent attack on our major population and industrial centers, it will be necessary that food resources be

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protected, conserved and provided for safe human consumption. Following an attack with nuclear weapons, accompanied by the use of biological and toxic chemical agents, the resulting situation would, in many respects, resemble peacetime disasters. Protected and unprotected foods in retail markets, delicatessens, eating and drinking establishments, catering facilities, home stocks and reserve supplies in warehouses will be buried in wrecked buildings, damaged by blast and fire and flooded with polluted water and sewage from a wrecked sewerage system. In addition, problems resulting from contamination by radioactive fallout, chemical and biological warfare agents will further complicate the disaster. Previous to or following an attack, the sabotage of milk, other foods and water supplies by biological agents and toxic chemical agents may occur. Targets will be the foods for vital industrial workers, military and governmental installations, mass evacuation centers, hospitals and other critical centers. Although the sabotage attacks may not result in high personnel fatalities, an adverse psychological effect will have been achieved.

Nuclear Weapons and Food Supplies

A. General Conditions

In an attack with nuclear weapons, damage to food supplies and food processing facilities through effects of blast and heat will be limited to the general area of the bomb burst. Foods in the immediate vicinity, if not destroyed outright by blast and heat, would become temporarily radioactive because of neutron bombardment from the fireball. The 1955 tests at the Nevada Proving Grounds with small weapons have demonstrated that food in glass or metal containers, located within 1/4 mile of the bomb burst, became intensely radioactive but could be used several days after the blast. Dairy products became radioactive, probably due to their calcium content. The half life of calcium is much longer than that of sodium or potassium, and this characteristic would delay the useability of dairy products. Furthermore, calcium is stored in the body. Sea foods also became radioactive. Glass food containers were particularly affected probably due to the formation of radioactive sodium.

Over great areas, foodstuffs, just as other materials in the environment, will be subjected to the fallout of radioactive materials originating directly or indirectly from a nuclear weapon. Domestic animals and man are equally sensitive to the effects or radiation but livestock, if on pasture during the fallout, would be more vulnerable to fallout effects. Unless remedial measures are taken during the emergency period, vast quantities of animals and animal food products might be lost. Preventive measures to avoid ingestion of foodstuffs contaminated with radioactive material therefore assume importance from a public health viewpoint.

B. General Courses of Action

Monitoring will identify areas free from radioactive fallout. Uncontaminated food and milk in these areas will receive early clearance for consumption. Serious problems may arise in areas showing presence of appreciable fallout material. Questions regarding the freedom of specific foodstuffs from radioactive contamination will arise on every hand. Within the limits of monitoring facilities, arbitrary maximum levels of radioactivity of food and

water will have to be used as a basis for determining their fitness for human consumption. Such a suggested scale of radioactive limits is presented in Table 1.

It is emphasized that these are not peacetime permissible levels of radioactivity for either short or long term consumption. Responsible officials can utilize these values during periods of emergency with the conviction that water and foods with radioactivity falling below these limits can be used with no real hazard. The beta-gamma values do not apply for periods later than one month after a nuclear explosion, since the longer lived radioactivity remaining carries an increasing hazard for human consumption.

It can be anticipated that the monitoring services available will be completely inadequate for direct evaluation of all food and milk originating within a fallout area during the first few days after an attack. Much thought is being given to this difficulty. An approach to this problem is emerging in which reliance is placed on the findings of area monitoring as a basis for deciding whether milk and other foods may be consumed immediately or if temporary or permanent embargo must be placed on their use. If the area monitors find lower gamma intensity levels than some maximum figure in roentgens per hour "x" hours after the bomb burst, then food originating within that area would be arbitrarily considered acceptable for human consumption. If area monitoring shows higher gamma intensity levels than the arbitrary tolerance level, then temporary or permanent embargo would be placed on use of foods originating within the area. Area monitoring information not only would approximately define the hazardous areas but it also would delineate the relatively safe areas; the safe areas would probably constitute the majority of the nation's rural land, and thus prevent unnecessary restrictive measures and loss in food production due to the lack of guide-lines.

C. Fallout Effects on Livestock and Animal Products

The effects of fallout on livestock and animal food products has been evaluated by Wolff.[2] He indicates that locales affected by fallout could be placed in one of four designated areas depending on the reported levels of gamma intensity in the area corrected to H + 7 hours at three feet from the ground. In this hypothetical evaluation, fallout is assumed to have begun 3.5 hours after the blast. These area classifications are: Area I - less than 6r/hr; Area II - 6-12r/hr; Area III - 13-30r/hr; and Area IV - greater than 30r/hr.

With a serious contaminating fallout, external and internal radiation hazards would exist simultaneously. During the first 24 to 72 hours following the fallout the external radiation will be the major concern. If there is sufficient warning time before the fallout begins, animals should be sheltered wherever feasible. After the fallout begins, protection of livestock should be ignored until monitoring data defines the human risk. The gamma radiation would contribute primarily to any resultant radiation illness and death due to its penetrating whole body exposure. The beta radiation, because of its limited penetrability, would contribute to the total radiation exposure by producing beta burns on skin areas to which fallout has adhered. The ingestion of contaminated feed would result in internal radiation exposure which would contribute to the total body exposure.

D. Marketing Animal Food Products During Emergency Period

Livestock survival is primarily affected by external radiation but after the first 2 to 3 days following the fallout, the hazards resulting from internal radiation will become important. The internal hazards to the animal per se is not the critical factor but rather the potential internal hazard to man resulting from ingestion of food products from livestock pastured in heavy fallout areas. The restrictions recommended for the marketing of food products derived from animals on contaminated pastures would not apply to the products derived from stabled animals in the same areas, provided they were receiving feeds with minimal fallout contamination. Most stored feeds, such as baled hay, silage, and grain, should be assumed safe for livestock feed.

Dairy cows, unfortunately, have the ability to concentrate various radioactive fission products in the milk following ingestion of contaminated feed. This fact plus the dietary importance of milk to infants and children and their greater susceptibility to radiation effects result in milk being the most critical animal food product.

Milk for manufacturing purposes should originate in a milk shed, if grazing is practiced, within the confines of Area I, that is less than 6r/hr. Under these conditions at least 30 days should elapse before the manufactured dairy product is marketed, thus permitting decay of the shorter lived radioactive substance in the milk. In the case of fresh bottled milk, even Area I may not be a safe milk shed and it is recommended that milk intended for the bottle or package market, originate in a milk shed, if grazing is practiced, where the H + 7 intensity is less than 0.3r/hr.

Meat animals, including poultry, would not metabolize fission products into the edible carcass parts in nearly the concentration that occurs with milk. It can be assumed that during the emergency period all food products derived from grazing animals in Areas I and II would be safe for human consumption. Those products from Areas III and IV will contain varying amounts of metabolized contamination and should be considered suspect and held in storage for a post-emergency radiological evaluation.

The above approximations and courses of action are proposed by Wolff as tentative guidelines and apply to the problems which might arise during the 15 day post detonation period. As early as possible after the emergency period, more accurate methods and controls should be utilized and all livestock grazing should be confined to pasture areas, the intensity of which does not exceed that originally defined as Area I.

E. Decontamination and Restoration of Food Supplies

If food is scarce, civil defense activity will include decontamination, salvage, and restoration of food supplies and facilities for preparing and serving food. Many foods will be available for immediate use, even within areas having dangerous radioactivity levels. These foods will be usable because of the protection from radioactive fallout afforded by packaging materials. Suggested decontamination procedures are indicated in Table 2.

It must be remembered that radiological decontamination is not a chemical reaction in which some neutralizing substance is added, with destruction of radioactivity. Decontamination is only a cleaning operation in which radioactive material is removed from places where it is or will be dangerous to places where it will be less dangerous. Cleaning may be accomplished by trimming outside layers of the food, with water, steam, detergents, complexing

agents, organism solvents, acid mixtures, caustics, abrasion, flame equipment, vacuum cleaning, and the like. The choice of the decontaminating agent and method will be determined by the nature of the surface to be decontaminated and by the availability of materials and equipment. Dangerous waste solutions should be buried in an unoccupied area which is appropriately identified.

Contamination of Foods with Chemical Warfare Agents

A. General

The contamination of foodstuffs by chemical warfare agents will create certain health hazards to the population and complicate salvage and decontamination problems. Food producing livestock, field crops, protected and unprotected foods and supplies may become contaminated by exposure to the vapor, aerosol sprays, liquid splashes of chemical warfare agents.

B. General Course of Action

If an attack with a chemical warfare agent or sabotage is suspected, all livestock, crops and foodstuffs in the area should be temporarily embargoed. Contaminated foodstuffs should be handled only by those trained in the detection and decontamination methods and equipped with proper protective masks and clothing. The courses of action to be taken will be based on results of the information obtained by trained personnel who have available CW test kits for field use.

C. Use of Milk, Livestock and Field Crops Exposed to CW Contamination

Until transportation routes can be repaired and food supplies replenished, it may be necessary to utilize the milk, livestock and crops in the contaminated area in mass feeding.

D. Decontamination Procedures

Until the type, extent and area of contamination have been determined, no foodstuffs should be used. Decontamination of food containers heavily contaminated with the toxic gases is dangerous. Unless food is scarce or large amounts are involved, the foods should be buried or burned in an uninhabited area.

Practical decontamination procedures for use under field or emergency conditions are (4):

1. Airing the foods for 24 to 48 hours.
2. Washing with water, dilute alkali (2% solution of sodium bicarbonate), 2% soda ash solution or bleach slurry.
3. Peeling and discarding the outer contaminated layers of the food.
4. Cooking the food (boiling water containing 2% sodium bicarbonate) for 1 to 2 hours.

Problems Resulting from the Use of Biological Warfare Agents and Deterioration of Sanitation Measures

A. Vulnerability

The reported incidence of foodborne disease in the U.S. under normal conditions during the 17-year period from 1938 through 1954 was approximately 4,700 outbreaks involving about 177,000 cases and resulting in 600 deaths. At best, no more than 10 to 25% of the true number of outbreaks were reported. These statistics will be magnified greatly with the use of biological agents overtly or in sabotage attacks, the contamination of foodstuffs with polluted water and sewage following the blast and fire of a nuclear weapon, and the deterioration of food sanitation safeguards attending mass evacuation or bombing disasters.

B. Rapid Detection of BW Agents in Foods

Following an attack in which contamination of food by BW agents seems probable, as indicated by presumptive evidence of sabotage or dispersal of an aerosol, all suspected foodstuffs should be temporarily embargoed until the kind, amount, and extent of contamination can be assessed. Unless there is an extreme shortage, it would be best to burn or bury all lots of foods found to be contaminated.

There is need for the development of more practical and rapid methods for the detection of biological warfare agents in or on foods and on food equipment surfaces.

C. Food Protection

Commercial canning procedures, in which heat processed foods are hermetically sealed in impervious containers, will protect the food from external contamination and subsequent spoilage. Although the outside of the container may become contaminated, the food inside should be protected, providing the package seams are not ruptured and no "pin holes" or channels have developed. The degree of protection afforded foods will vary with the packaging material and type of package. Until proven otherwise by laboratory and field tests, it must be assumed that all packaged foods exposed to BW contamination will have to be decontaminated before opening for use.

In the absence of suitable processing and storage facilities, or other normal sanitation safeguards, consideration may be given to the emergency use of antibacterial agents, such as hydrogen peroxide and antibiotics, as milk and food additives to minimize spoilage and limit occurrence of foodborne disease outbreaks.

A safe water supply for cooking, dishwashing and general sanitation will be of utmost importance. Waters and ice of unknown quality should not be used by the operating eating establishments and welfare organizations.

Attention must also be given to the protection of food supplies to prevent insect and rodent infestation or scavenging by animals.

D. Decontamination of Food Containers and Equipment

Salvage operations should be supervised by personnel with a knowledge of the physical and chemical properties of the decontaminating materials and methods to be used.

The following decontamination procedures and agents are suggested:

1. Heat
2. Chemical Disinfectant Solutions
3. Chemical Gaseous Disinfection
4. Physical Decontamination
5. The persistence of biological agents varies with the kind of organism and the environmental conditions. The effects of sunlight, oxidation, moisture, temperature and dilution all serve to decrease the number of viable microorganisms. If conditions permit, aging and weathering will reduce greatly surface microbial contamination in a few days.
6. Foods contaminated with BW agents should be buried under supervision to avoid the spread of the agents and reinfection of the surviving population.

Sanitation Problems Associated with Mass Feeding

A. Responsibilities

The emergency welfare services will have the primary responsibility for mass care, emergency housing, feeding and clothing, but these services will look for assistance to the engineering and health services to provide: (1) quality control or food safety at the source, (2) protection of foods in transit and storage, (3) sanitation controls at mass feeding centers, (4) safety of water supplies, (5) waste problems, (6) insect and rodent control, (7) recruitment, training and supervision of sanitation personnel, (8) assistance in site selection and other problems.

B. Reception Areas

Consideration will have to be given to the safety of greater quantity and varieties of food. First priority must be given to providing a safe supply of water and milk. Preplanning should include the organization and training of sanitation squads drawn from either local or evacuee personnel. However, these squads should be permanent and to be effective will need to have more training. It is anticipated that in reception areas, professionally trained sanitation personnel, resident in the area or brought in from the outside, will have to be made available to assist the welfare services.

SUMMARY

It is obvious that the sanitation services of a health unit cannot operate in a vacuum or void. Pre-planning and coordination with other services, such as, welfare, supply, transportation, communication, police and others will be required. It is necessary to more clearly define the areas in which public health workers will be of service in specific assignments relating to food safety and mass feeding.

Engineering Implications of Milk and Food Problems in Civil Defense

The sanitary engineering aspects of civil defense must, of necessity, include certain responsibilities relating to the supply of milk, food, and water.

The maintenance and operation of these basic resources under emergency conditions requires specialized engineering training. All sanitary engineers, however, should possess general knowledge of procedures for meeting conditions resulting from attack by nuclear weapons, or from sabotage by the use of biological or toxic chemical agents. The public health considerations involved in these aspects of civil defense are reviewed in this paper. In addition, there is provided a digest of the recommended courses of action, of suitable protective methods, and of procedures for decontamination.

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Credit

This research report, which is one of a series of professional contributions by the Committee on Sanitary Engineering Research,

E. R. Hendrickson, W. T. Ingram	Air Pollution
H. R. Amberg	Stream Pollution
F. G. Paloscay	Sewage
C. H. Hull	Water
H. A. Faber	Public Health
R. Stone	Refuse
Chairman, N. L. Nemerow	Industrial Wastes

has been prepared by the Public Health Section.

H. A. Faber, Head

Table 1

SUGGESTED EMERGENCY LEVELS FOR FOOD AND WATER
IMMEDIATELY FOLLOWING A NUCLEAR EXPLOSION(1)

CONSUMPTION PERIOD	BETA ACTIVITY LEVELS		ALPHA ACTIVITY LEVELS	
	MICROCURIES PER CC	DISINTEGRATIONS PER MIN PER CC	MICROCURIES PER CC	DISINTEGRATIONS PER MIN PER CC
PREFERABLE RISK				
10 days	3.5×10^{-3}	7.7×10^{3}	2×10^{-4}	440
30 days	1.1×10^{-3}	2.6×10^{3}	6.7×10^{-5}	145
ACCEPTABLE RISK				
10 days	9×10^{-2}	2×10^{5}	5×10^{-3}	1.1×10^{4}
30 days	3×10^{-2}	7×10^{4}	1.7×10^{-3}	3.7×10^{3}

Table 2

Salvage of Typical Foods[3] [4]

Kind of food of package	Possible salvage procedure in case of -		
	Contamination with radioactivity	Blast and fire damage	Water damage (pollution)
Perishable: Fresh fruits & vegetables, fish, poultry	Remove outside portions of lot containing most radioactivity. If remaining contamination is less than emergency tolerance, release interior portions. Washing of fruits & vegetables may be of value.	Look for contamination with poisons. If pathogens are present sterilize before using.	Wash to remove surface contamination. Cook to kill bacteria.
Non-perishable: Dried fruits & vegetables, flour & grains, bulk sugar stocks.	Remove outside portions of lot containing most radioactivity. If remaining contamination is less than emergency tolerance, release interior portions. Washing of fruits and vegetables is not feasible.	Look for contamination with poisons. If pathogens are present sterilize before using.	Prompt sterilization & use of fruits and vegetables. Remove flour and grain which is not caked. Cook before using. Refine sugar.
Cardboard and paper containers	Remove outside portions of lot containing most radioactivity. If remaining contamination is less than emergency tolerance, release interior portions. Remove dust by brushing. Remove outer wrappers.	Look for contamination with poisons. If pathogens are present sterilize before using.	If salvage attempted, sterilize food from water-damaged containers before it is consumed.
Canned goods: Hermetically sealed cans.	Wash outside of container with detergent, or remove radioactivity by brushing. Interior portions of stacks may be relatively free of radioactivity.	Look for & destroy cans with ruptured seams or closures. Remove abnormal cans. Look for spoilage from thermophilic organisms.	Sterilize surfaces of cans. Watch for pinholing of metal. Use damaged stocks promptly.
Containers with screw caps friction type lids, etc.	Wash outside of container with detergent, or remove radioactivity by brushing. Interior portions of stacks may be relatively free of radioactivity. Test contents before releasing for use.	Look for & destroy cans with ruptured seams or closures. Remove abnormal cans. Look for spoilage from thermophilic organisms.	Difficult to remove contamination from beneath or around closure. Sterilize foods before using.

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IMPORTANCE OF GROUND WATER IN OUR NATIONAL ECONOMY[1]

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(Proc. Paper 1271)

SYNOPSIS

The use of water derived from underground sources by means of wells has been increasing greatly in recent years. Though ground water meets only about 15 per cent of the Nation's total requirements for water, it is an important resource upon which a large segment of our population depends. There is no national ground-water problem as such, but there are numerous widely scattered problem areas, each differing more or less markedly from its neighbors because of distinctive geologic and hydrologic environment and other local factors. Typical ground-water problems in six areas are reviewed and it is concluded that the solution of each must rest on adequate knowledge of the character and capacity of the ground-water reservoirs involved and on comprehensive planning for the best use of the available ground-water supply. To solve many of the major problems will be complex and expensive, and practically all will require considerable public education and support if an adequate solution is to be achieved.

INTRODUCTION

As has been clearly stated in the recent report of the Presidential Advisory Committee on Water Resources Policy, there is no "national" water problem. Instead, there are individual problems distributed Nationwise, relating to the use and development of the water resources, which are different in both character and severity from place to place. It is appropriate, therefore, at a national meeting of this Society, whose professional and technical activities are so intimately concerned in so many ways with the availability of water and the means of utilizing it in support of the economy of the Nation,

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that attention should be devoted to the importance of ground water and ground-water problems in our National economy.

The great attention that our water resources have been receiving in recent years is substantial evidence that both public and private interests are becoming increasingly aware that continued economic expansion is limited by the availability of adequate water supply of good quality. There are no such things as inexhaustible supplies of potable water, and it is now well recognized that stock must be taken of the water resources that are available in order that their use and conservation may be wisely planned to meet the increasing demands of modern industrial civilization.

The conclusion is obvious that there is no single solution that can be applied to all the water problems of the Nation. Rather, a wide variety of solutions are required, each adapted to the particular conditions and demands of the area under consideration. There are shortages of water supply in many places; in other places, it is obvious that, at present rates of withdrawal or at rates that are in prospect, shortages will occur in the not too distant future; and in still other areas where ground water is a major source of supply, it is being exhausted by being withdrawn from the underground reservoirs at a rate far greater than the rate of replenishment. In such areas the water is, in effect, being mined. Though numerous water problems exist throughout the Nation, in both humid and arid regions, their multiplicity does not mean that we are "running out of water," as is claimed in some quarters.

Despite some pessimistic predictions that we will not be able to meet the large prospective demand for water, the situation, as a whole, is not dark. We are one of the best watered nations in the world, and, though our population is growing and industry is expanding, our technical knowledge of our water resources also is expanding, though admittedly not in keeping with the demands that are being made for water-resources information. However, with the expansion of our knowledge, our problems are being more clearly defined and new solutions are constantly being developed. It must be recognized that in many cases the solution of the problems will involve expenditures of large proportions, but, as in many problems of modern industrial civilization, the solution is generally a matter of economical rather than physical feasibility, and the necessary economic adjustments will be made wherever failure to make them would result in a still greater cost to the community and the Nation.

Importance of Ground Water Indicated by Size of Industry

Some idea of the importance in our national economy of the water developed from the ground by means of wells may be gained by taking a look at the well-drilling industry. The Water and Sewerage Industry and Utilities Division of the Business and Defense Services Administration of the U.S. Department of Commerce provides the best estimate available of the size of this industry. Although no complete census has been made, it is estimated that there are more than 9,000 well-drilling contractors in the United States. This figure has been adjusted to make allowance for a relatively large number of part-time operators who construct a few wells each year, but whose major occupation is some other activity.

It is estimated that the 9,000 well-drilling contractors operate about 22,000 drilling machines and that they employ about 50,000 workers. Estimates of

the number of wells constructed each year range from 460,000 to 600,000. The latter figure seems somewhat high in relation to estimates of materials used in the construction of the wells and equipping them with pumps. It seems reasonable, therefore, to estimate the total number of wells currently constructed each year as approximately 500,000. The new construction involves expenditures estimated at $475,000,000 in 1953. This number does not include dug wells and small driven and bored wells put down by hand, of which a considerable number are still in use in some areas.

W. L. Picton of the Water and Sewerage Industry and Utilities Division of the Department of Commerce estimates that the total number of wells in operation has grown from an estimated 7 million in 1900 to approximately 13-1/2 million today, and that, by 1975, the number may reach 16 million. In viewing these figures, it should be kept in mind that a substantial portion of the annual construction—perhaps about 300,000 of the 500,000 wells—represents the replacement of wells that have reached the end of their economic life, so that currently about 200,000 wells are added to the total inventory each year.

The total requirements of the Nation for water have grown tremendously in recent years. It is estimated that in 1900 the aggregate use of water was about 40 billion gallons a day, of which amount 7 billion was supplied by wells. The total use in 1955 is estimated at about 260 billion gallons a day, of which about 40 billion was obtained from underground sources. On the assumption that the national economy will expand and that present trends will continue, it is reasonable to predict that normal growth in population, nonfarm home-building and movement of population to suburban areas, modernization of farms, industrial growth, and the demand for water for irrigation will bring about a large increase in the use of water from wells. By 1975 the quantity developed from the ground may reach 69 billion gallons a day, according to Picton.

The well-drilling industry is not a big one if compared with some other segments of our economy, but its importance looms large when the results of this activity are viewed in relation to our present mode of life. It has been a significant factor in the decentralization of many industrial operations, in the movement of population from cities to suburban and rural areas, and in the expansion of irrigation in both humid and semiarid regions.

Problem Areas

As a sort of penalty that must be paid for the many benefits accruing from the use of ground water, its development in many sections of the country has brought about certain problems. Perhaps as good a way as any to bring out the varied and individual nature of the problems would be to look at a few of them briefly. You will note that they all have this point in common: There is danger that there will not be enough water of the desired quality or temperature, at the specific points where it is desired, at a cost that is convenient or feasible to pay. But there the universal agreement ends. On other major points—the local geology and hydrology, the local economic situation and pattern of water needs, and the local background of water law as it influences the ways in which the right to use water can be acquired—each problem area will be found to differ more or less markedly from its neighbors.

The West has its full share of major water problems. One might say more

than its share, if it were not that the present situation in the West is the inevitable result of regional hydrology and national history, and so can hardly be said to be surprising or unexpected. It was the Geological Survey's second Director—Powell of the Colorado—who long before the turn of the century made what has proved to be one of the most astute observations and predictions of all time. In the face of the most optimistic claims of promoters as to the West's fertility and productiveness, he made the flat statement that there would never be enough water to take care of all the irrigable acreage—indeed, there would be water enough for only 1 to 3 per cent of the West's total area. In making this prediction he neatly bracketed today's irrigated acreage in the West—roughly 2 per cent of the total area—and it is becoming obvious today as never before that we have nearly reached the limit of practical irrigation in the West. New irrigation projects are already being balanced by the retirement of farmland as the result of depletion of water sources, deterioration of soil, growth of suburbs, and highway and industrial construction. According to Secretary of the Interior Fred A. Seaton, writing in the Reclamation News for November 1956, these and other causes have removed more than 5.5 million acres of land from farm use, a substantial part of it being irrigated land, during the past 3 years.

Roswell Basin

One of the problem areas of the West is the Roswell artesian basin in New Mexico, where the writer more than 30 years ago took part in one of the pioneer efforts in quantitative ground-water hydrology. Though differing from the typical Western valley in geology and hydrology, the Roswell basin shares with many others the characteristic of a perennial natural water supply which, though large, can be overdeveloped in such a way that its usefulness in supporting the economy of the region can be impaired.

The basin lies on the easterly dip slope of the Sacramento Mountains. Its history goes back to the time when the mountains themselves were formed by the uplift and eastward tilting of a huge block of the earth's crust. The broken western edge of the block forms the steep west face of the mountains, overlooking the basin into which the adjacent block foundered—the Tularosa Basin. As the Sacramento block rose, streams flowing from its crest carried erosional debris eastward, forming a part of the vast alluvial apron whose remnants are now known as the High Plains. Subsequent uplift and erosional and solutional activity resulted in trenching of the alluvial apron and isolation of the High Plains from the mountains. In the Roswell area the separation of mountains and plains was achieved by the southward-flowing Pecos River. The river probably once flowed somewhat west of its present course, at a higher level, and has reached its present position by gradual retreat down the dip slope of the beds along the flanks of the Sacramentos. As the river shifted eastward it cut down, and with its westward tributaries, it stripped away still more of the younger sedimentary rocks of the mountain block and eventually exposed the soluble limestone which was to become the aquifer of the Roswell basin. The river retreated eastward down the dip slope faster than it could remove all the rocks above the limestone. The result: a belt of limestone exposed on the east flank of the mountains, east of which belt the limestone strata dip beneath the still unremoved shale, gypsum, and sandstone of the overlying beds.

As the mountains rose greater precipitation was induced along their crest,

and as larger areas of the limestone strata were exposed by erosion progressively larger quantities of water began to enter through cracks and move down the dip in the limestone. Much of the water returned to the surface in springs at the retreating featheredge of the younger rocks, but some of it passed beneath those rocks, gradually enlarging the crevices as it moved eastward. Farther down dip the openings in the rocks became so few and other outlets so remote that the water found it easier to seep upward through the overlying rocks and into the alluvium along the Pecos River than to move farther east.

Thus, in time, the ground-water system developed to the stage that existed when man discovered the artesian basin, water entered the outcrop of the limestone on the mountain slope, passed beneath the confining beds of younger rocks in openings which the water had created or enlarged by solution, seeped upward under the artesian head provided by the altitude of the recharge area, entered the alluvium deposited along the river, and eventually discharged into the river or was captured by the thirsty flood-plain vegetation.

The hydrologic makeup of the Roswell basin has been described in more detail than will be given to other areas mentioned because the description emphasizes an important point: the availability of ground water in any area is governed by the hydrology of that area, and the hydrology of that area is different in one or more major respects from the hydrology of all other areas, no matter how similar they may be otherwise.

And so, here was a place where man soon found that he could obtain a bountiful supply of water by natural flow by drilling into the limestone that was made cavernous and permeable by circulating water. Some wells flowed thousands of gallons a minute, and it is not surprising that the early water users considered the source inexhaustible. But as development mushroomed during the period 1905 to 1915, it soon became apparent that there was a limit to the water supply. As well after well was drilled, the artesian head began to decline; the yields of all the wells diminished, and the wells on higher ground stopped flowing altogether. To make matters worse, poorly constructed wells or rusted-out casings in the older wells permitted water to leak directly into the alluvium at a rate greater than was permitted by the relatively tight rocks through which the water had to discharge under natural conditions. To continue the irrigation of lands previously supplied with flowing artesian water, it was necessary to install and operate expensive pumping equipment. Ultimately, the decline in head became so great that many farms situated along the western side of the basin at higher elevations had to be abandoned because crop returns were inadequate to meet the high cost of pumping the artesian water.

Thus, early in the twenties, the economy of the whole area began to change. It was then that the Geological Survey was called on to investigate the basin in cooperation with the State Engineer, to determine how much water could be developed perennially so that the State would have a basis for the regulatory action that had become inevitable. The Survey's investigation formed the basis of a State law, enacted in 1927, declaring waters "in underground streams, channels, artesian basins, reservoirs, or lakes, the boundaries of which may be reasonably ascertained by scientific investigations or surface indications" to be public waters and subject to appropriation for beneficial uses under the existing laws of the State relating to appropriation of waters from surface streams. In order to test the constitutionality of this legislation, two suits were brought in the district court, and in 1929 the State Supreme Court rendered a far-reaching opinion upholding the law in principle,

but declaring it unconstitutional nevertheless. The objection was strictly a technical one: the law incorporated an existing statute but referred to it only by title instead of quoting it in full, as required by the State constitution. A new act, drawn so as to overcome the objections advanced by the court, was passed in 1931. This law has stood up against all legal attacks and has served as a model for similar laws in several other States. It provides a basis on which the use of water can be controlled so as to hold the use within safe limits.

In the atmosphere provided by the knowledge of the basin's water resources and by the control of development under the State law, the local economy was strengthened. Leaky wells were repaired or sealed to reduce the loss of water through them. Pumping of ground water from the alluvium, recommended by the Survey as a means of providing water for additional acreage without greatly increasing the draft on the artesian reservoir, was expanded. Other recommendations of the investigation also were placed in operation, and the agricultural economy of the basin was again placed on a sound basis.

It would be gratifying to report that here, indeed, is one area without a water problem, but this is not true. Under the stimulus of high prices for farm products during and since World War II, the withdrawal of ground water from the basin as a whole has increased persistently, in spite of the control afforded by the State law. This increased pumping has created a situation in which ground-water levels have declined to new lows, saline water is encroaching from the east, and the agricultural economy of the basin is again being threatened.

An adjudication of the water rights in the artesian basin is currently in progress. This procedure, when completed, will establish the priority of rights and doubtless will eliminate from the irrigated area some acreage for which valid water rights do not exist. However, the increase in the use of ground water for irrigation has been so great that the reduction in draft resulting from the elimination of acreage having no valid water right probably would not be sufficient to bring the remaining draft within the safe yield. Further reduction in draft will undoubtedly be necessary, either through limiting use of water on acreage of low water-right priority, through a reduction in the use of water on water-right land, or through the relentless action of economic and hydrologic law.

Doubtless, as time goes on, some of the water will be diverted to certain industrial or other uses giving a greater economic return than irrigation. Nevertheless, the goal of economic security will be reached only through recognition of the nature of the problem by both water users and the general public, and hard work and cooperation on the part of all over a period of years. No permanent solution of the ground-water problem here will be successful unless it is based upon the widespread recognition and acceptance of the fact that the use of water cannot be continued in the long run at a rate that is substantially in excess of the perennial yield of the ground-water reservoirs.

The Roswell basin provides a sobering example of an area which, though blessed with a large natural ground-water supply, though possessed of a hydrologic setting favorable to a permanent water supply, and though located in a State having a model water law, requires constant vigilance and persistent effort. Many other areas have less in the way of assets and more in the way of liabilities, and yet must try to achieve a comparable solution.

Southern High Plains

Not far from the Roswell basin is one of these areas where the water problem is even tougher than it is in the basin—the High Plains. Here is an area of large natural ground-water storage but negligible natural recharge—in other words, an area whose ground water is like coal or oil in that to use it on a large scale is to mine it out for good.

The High Plains, as mentioned previously, are a vast apron of alluvium spread out by the eastward-flowing streams that carried water and erosional debris from the Rocky Mountains. After the apron was laid down it was isolated from its mountain source by erosion, and it has been divided into several large segments by the major streams crossing it. Thus today the High Plains consist of a number of large areas each of which depends for its ground water on local precipitation. Because precipitation is light—20 inches or less in much of the area, because evaporation is high, and because there is a tight subsoil in much of the Plains, nearly all the precipitation is discharged by evaporation and transpiration. Little runs off, and an average of only a fraction of an inch per year is able to penetrate to the zone of saturation in the lower part of the alluvial apron, resting on the tight rocks below.

The southern High Plains of eastern New Mexico and western Texas, south of the Canadian River, are a vast underground reservoir which stores something like 200 million acre-feet of water but which is replenished at a rate probably less than 100,000 and perhaps less than 75,000 acre-feet per year.

Domestic, stock, municipal, and industrial water supplies in the High Plains are obtained almost wholly from wells, as surface-water supplies are almost nonexistent. Until the 1930's however, irrigation wells were few. Then the droughts and crop failures of the early thirties made it clear that precipitation could not be depended on for successful crop growth, year after year. Hence, since about 1935, the development of ground water for irrigation in the southern High Plains, particularly in Texas, has accelerated at a rate unmatched elsewhere in the Nation. Each year a new record is set in the number of wells in use and quantity of water pumped. Pumping in the High Plains has been the largest single factor in the tenfold increase in ground-water pumpage in Texas in the last decade, to a total for the State of more than 7 million acre-feet per year, second only to California. And, in the High Plains, this water is being mined, for the natural discharge at the edges of the Plains continues unabated and will not be affected substantially by the pumping for years to come. The increase in pumping in the New Mexico section of the southern Plains has been less spectacular, but it is still impressive.

What kind of situation faces the users of ground water in the southern High Plains? At first many of them refused to believe that the water supply was not inexhaustible. Their contention was strengthened by the heavy rains of 1941, which raised the water table to record heights and gave every impression of abundance. But the rains of 1941 were the heaviest, or close to the heaviest, on record in much of the Southwest, and they will not be repeated on the average for 50 or 100 years. In the face of evidence that most recharge on the High Plains occurs in such years, and little or no recharge in average or dry years, it is easy to realize that the recharge of 1941 was but a drop in the bucket compared to the pumpage since that year.

Thus the well owners of the High Plains have gradually become aware that they are pumping from storage. It is too early to say that they have become

adjusted to the idea, for to date no one knows exactly how the region is going to meet the problem of depletion of stored ground water. Obviously, if events are allowed to run their course without interference, irrigation from wells will have to be abandoned in areas that will begin at centers of heavy pumping and will grow outward from those centers as the water is depleted in the outlying areas. Owners of land in the affected areas will have to turn to other activities—dry farming, grazing, or others—or will have to sell out and move. Areas wherein the saturated thickness of the water-bearing formation is small will be the first to experience difficulty in obtaining water in quantities sufficient for irrigation, whereas those areas where the saturated thickness of the formation is large may have adequate water for heavy irrigation for a relatively much longer period.

What are the possible solutions? Storage and distribution of the available flow of the major streams might help locally but could not replace the water from wells—there is not enough surface water. Importation of large quantities of water from the more humid regions to the east is still a dream of the future, as is conversion of whatever saline water may be present. Artificial recharge of some of the water that now ponds and evaporates has some promise, but how far it can go toward meeting future water demands as large as those of today is still an unanswered question. Economy in the use of ground water in irrigation can help at least to some extent, and efforts in that direction are widespread.

The problem is complicated by the fact that a single aquifer is divided by a political boundary into two parts, in each of which the approach to conservation and allocation of ground water differs widely from that in the other, because of basic differences in legal philosophy. In New Mexico the appropriation principle is followed. The doctrine of prior appropriation is based on the assumption that the water supply is perennial, whereas the water supply in the High Plains is exhaustible so far as large demands are concerned, unless artificial recharge can be practiced on a scale far larger than seems possible under present conditions. Thus, strictly speaking, it would be impossible to acquire rights to the use of water at a total rate greater than the average recharge, which would mean that only domestic and very limited municipal and other uses could be sustained perennially. The State Engineer allows the appropriation of additional water until, in his judgment, a point is reached where existing developments can continue successfully for a period of 40 years before the water is depleted to the point that the developments will become uneconomical. Further applications are denied because they would shorten the useful economic life below the figure of 40 years selected as the minimum period necessary to compensate the individual, the community, and the State as a whole for the exhaustion of this natural resource.

Texas law recognizes the principle of prior appropriation to some extent for surface streams and so-called underground streams but not for so-called percolating water, which includes the water beneath the High Plains. A specific statute provides for the recognition of underground reservoirs of percolating water and for the formation of water-conservation districts which have the power to establish regulations governing the conservation of ground water. One such district covering essentially all the southern Plains in Texas has been formed and is active in promoting conservation of water, by eliminating wasteful use, and research on artificial recharge of storm water.

Here, too, is an area whose problem will be solved only by dint of strenuous effort over a period of years, using means that have never been used

before because a problem of this kind and on this scale has never been faced before. If the problem is solved successfully, as it must be, man will have made a significant addition to his bag of tools for handling analogous problems elsewhere.

Miami, Fla.

The humid East has its water problems too. As an initial example we might select an area that is about as humid as the East gets—the Miami area of Florida, where the average annual rainfall is approximately 60 inches. Though the climate is warm and the loss of water by evaporation and transpiration is high, the area is underlain by soil and rocks so permeable that a large amount of rainfall escapes evapotranspiration and recharges the groundwater body. The permeability and transmissibility of the sand-and-limestone aquifer are among the highest known in the United States and the world. For example, pumping at a rate of more than 40 million gallons per day from the city's Miami Springs—Hialeah well field produces a cone of depression in the water table that is less than four feet deep. The maximum yield of the more productive wells throughout the area is thousands of gallons per minute and is determined more by practical considerations of pump and casing size than it is by the yield of the aquifer.

Where, then, is there a problem in this ground-water man's Paradise? The answer lies in salt water and in drainage canals. Much of the metropolitan Miami area lies on a low coastal ridge and is not subject to widespread flooding, but in certain coastal areas and in the hinterlands flooding occurs during parts of most years so that the land must be either drained or filled to make it usable for ordinary development. The drainage canals have lowered the water table, and with it the head that formerly kept the aquifer so full of fresh water that fresh springs discharged along the shore. With the lowering in head, salt water has moved inland, both on a broad front along the shore and in the form of underground tongues along the canals. Salting of near-shore wells, including those of the city's Coconut Grove well field, brought the problem to public attention in the early thirties. Not until 1939, however, did the problem become acute. In that year drought conditions and a resultant low canal flow permitted salt water from Biscayne Bay to advance upstream in the Miami Canal to a point beyond The Miami Springs-Hialeah well field. In response to the lowering of the water table caused by pumping, the salt water seeped out of the canal through its permeable bottom and sides and entered some of the public-supply wells. Meanwhile the landward advance of the underground salt-water tongues along the Miami and other tidal canals was accelerated. It was at this time that the Geological Survey's ground-water studies in cooperation with the State Geological Survey were broadened to include cooperation with Dade County, the cities of Miami and Miami Beach, and other agencies, and an intensive study of the hydrology of the Miami area and the rest of southeastern Florida was begun. The study showed that the Miami area will be able to count on a very large groundwater supply so long as salt-water encroachment in and along the tidal canals can be controlled. The principal means of control are adjustable dams placed in the canals, opened during normal and high water levels to allow the canals to perform their function of controlling the rise of the water table; closed during low water levels to halt the inland flow of salt water in the canals and to maintain a higher water table.

At present the salt-water situation is more or less under control, but a few troublesome spots remain. Some tidal canals are still uncontrolled, and the control structures in some others are too far upstream to be effective in restraining salt-water encroachment nearer the coast. Also, in recent years many new tidal canals have been proposed for low-lying coastal areas to obtain fill for reclaiming swampy land for homebuilding and to provide access to the Bay for pleasure boats. Each of these canals in effect would be an extension of Biscayne Bay, bringing salt water inland as far as the canal extends, and bringing with it the threat of additional salt-water encroachment. Appropriate control measures, where feasible, should be taken to safeguard the precious ground-water supply.

Long Island, N.Y.

The Nation's greatest metropolis has had a ground-water problem that is similar in some respects to that of Miami. New York City's water supply is obtained largely from upstate surface reservoirs, but ever since the early days a large share of the supplies of both domestic and industrial water on Long Island has been obtained from wells. Long Island accumulated much of the debris scraped up and carried along by the southward-moving glaciers of the Pleistocene epoch, some thousands of years ago. Advancing from rocky New England, the ice sheets picked up vast quantities of comminuted rock and dumped it, blanketing a ridge composed of eroded Cretaceous coastal-plain sediments. The glaciers built up a compound ridge of largely permeable glacial drift which forms most of what we now know as Long Island. This sandy drift, recharged from precipitation at a rate of something like a million gallons per day per square mile, makes of Long Island's 1,400 square miles an aquifer having a potential yield of many millions of gallons a day. Beneath the drift are Coastal Plain strata which include many beds of sand that form productive aquifers. These do not have a source of recharge of their own, as they depend on rainfall passing through the overlying glacial drift; hence, they do not actually add to the ground-water potential. However, they do provide a means of obtaining water in areas where the water in the drift is scanty or of poor quality. In fact, because of the possibility of contamination, there is a tendency to develop water supplies in the deeper aquifers; and nearly half the total pumpage now is taken from these older deposits.

Long Island's abundant ground water has long been recognized and utilized. Because the population is greatest at the western end, the ground-water withdrawal likewise has been concentrated there. The withdrawal, in fact, became so great that by the late twenties the water table had been drawn below sea level in a large part of Brooklyn, and the lowering extended to the shore in broad stretches and induced salt water to encroach. Recognition of the danger led in the early thirties to enactment of a State law placing control of ground water withdrawal on the Island, except that for agricultural purposes, in the State Water Power and Control Commission. As a result of regulations established by that agency, there has been a considerable reduction in ground-water withdrawal in the areas of heaviest pumping. Ground water used for air conditioning, but not contaminated by that use, is returned to the ground at the rate of tens of millions of gallons per day in what is the leading development of artificial recharge through wells yet undertaken. Little by little, public authorities have closed down or reduced the withdrawal from the well fields that formerly supplied public water in Brooklyn and Queens, substituting

water from the upstate sources. The water table in Brooklyn has now recovered to a point where salt water is no longer actively encroaching, or at least is moving at only very slow rates. Nevertheless, the demand for ground water of relatively low cost for numerous industrial and commercial uses in this congested area is bound to continue, so that it will be necessary to keep a close watch on the area to make sure that the ground-water situation does not retrogress. Meanwhile, the pumping of ground water for public and industrial supply and for irrigation has increased by leaps and bounds in Nassau and Suffolk Counties to the east, and great care will have to be exercised to prevent the creation there of problems similar to those which have plagued western Long Island. The maximum development of the ground-water resources of this productive island on a perennial basis will require perpetual vigilance, in view of the fact that the island is surrounded by salt water that is always ready to move inland in response to the lowering in head that accompanies the pumping of ground water.

Midwestern States

Ground-water problems of a different kind exist in parts of a large Middle Western area underlain by artesian sandstones of Cambrian and Ordovician age. The area in which the sandstones yield usable water includes eastern and southeastern Minnesota, western, southern, and eastern Wisconsin, northern Illinois, and northeastern Iowa. The strata cover the flanks of highlands of Precambrian rock in central Minnesota and Wisconsin, dipping to the south and west into Iowa and Illinois and to the east into the Michigan basin. From the areas where they crop out at the surface or beneath glacial drift they pass beneath younger Paleozoic rocks and the water in them becomes confined under artesian pressure. Though the sandstones are not comparable in permeability with the aquifers of the areas described previously, they yield from a few hundred to a few thousand gallons per minute to wells penetrating their combined thickness, which in places is more than a thousand feet, and they are accessible in many places where aquifers in the overlying younger Paleozoic rocks and in the glacial drift yield little water or water of poor quality.

The principal problem in the utilization of the water supply of these sandstones is one of economics. Because of the accessibility of the sandstones they have been widely developed as sources of municipal and industrial water supply. Even though they have been replaced as prime sources of the principal public water supplies in the larger metropolitan areas including Chicago, Milwaukee, and St. Paul-Minneapolis, they are pumped heavily for industrial use and for the public water supply of numerous suburban communities. Because the sandstones are not highly permeable in comparison with the Nation's better aquifers, because they have the small storage coefficient characteristic of artesian aquifers, and because the areas of heavy pumping are some distance from the principal areas of recharge, the water levels in the sandstones have declined greatly since the early days—by hundreds of feet in many places. Still, the sandstones have not been unwatered, as they lie deeper than even the depths to which the water levels have been drawn, and water can still be pumped so long as the user is willing to lift the water hundreds of feet. Eventually, the pumping rates in the areas of heavy development will stop increasing, or even decrease, as the cones of depression extend and merge and the hydraulic gradients toward the stabilized bottoms of

the cones become more gentle. Likewise, the pumpage will decrease as the cost of lifting the water approaches the cost of buying water from the public mains. Inevitably, there has been some hardship as well users who grew accustomed to low-cost water pumped from moderate depths have been forced to deepen wells, lower pumps, or even stop pumping altogether. On the other hand, the water has contributed substantially to the prosperity of this important industrial and farming region, and the only way to obtain it in the quantities that have been developed has been to tolerate the declines in water level. Meanwhile, in large areas between the centers of heaviest pumping, the water levels have not been drawn down far and water is still available with moderate lifts. For the maximum development of the available supply in these sandstones it is obvious that, where economically feasible, some of the water demand in centers of heavy pumping might profitably be met by pumping from these areas of moderate lift.

Throughout the area in which the artesian sandstones provide relatively large quantities of water, there are less extensive aquifers such as Paleozoic limestones, glacial outwash, and valley fill along recent and preglacial valleys. These are, in large part, undeveloped and few quantitative data on their capacity are available. However, these aquifers in some places yield substantial supplies and their availability for further development should not be overlooked.

Yazoo Delta of Mississippi

In reviewing examples of the variety and range of the Nation's groundwater problems, we might select an area not far from our meeting place—the Yazoo Delta of northwestern Mississippi. The Delta is an area of rapidly expanding development of ground water for irrigation. It is not now a serious problem area, but it is one in which problems are very likely to develop unless steps are taken reasonably soon to forestall them.

The Delta is a part of the prehistoric valley of the Mississippi River, which in this, its area of widest meandering, has wandered as far east as the present course of the Yazoo in Mississippi and as far west as the valleys of the Arkansas and Ouachita Rivers in Arkansas. During the Pleistocene epoch, at times when the climate became warm and the glaciers of the north melted back, the Mississippi carried vast floods of melt water, which scoured out a huge, nearly flat-bottomed trench scores of miles wide and a couple of hundred feet deep. The trench was partially filled late in the ice age or at the beginning of the Recent epoch with glacial debris, much of it coarse and well sorted. Later, as the Recent epoch wore on, the flow of the streams slackened and the coarse glacial outwash was covered by deposits of silt and clay.

Irrigation in the Delta was first practiced using water from streams, but about 1950 a rapid development of irrigation from wells began when farmers realized that beneath their feet was a huge reservoir of water that was available wherever one chose to drill, and that was more reliable than the surface-water supplies which diminished in periods of drought just when they were needed the most. By the end of 1955 nearly a thousand irrigation wells had been put down, and by the end of 1956 the total probably approached 1200. The pumpage in 1956 has not yet been tabulated, but it is estimated to have been about half a million acre-feet. Bolivar County in the west-central part of the Delta became the center of extensive development. Comparative

measurements made in 145 observation wells in the county show there was a mean lowering of water level of 1.1 feet in the two-year period from August 1954 to August 1956.

In 1956 about a third of the irrigation wells were used for the cultivation of rice. It is the silty and clayey subsoil that makes flooding of rice practical, but that same subsoil impedes recharge from precipitation and irrigation water. For the possible significance of this situation we have only to turn to the Grand Prairie of Arkansas across the Mississippi, where conditions appear to be somewhat similar. There the pumping of water for rice irrigation and the impeding of recharge by the tight subsoil have combined to cause depletion of ground water to the extent that means are having to be sought to replenish the ground water by artificial recharge through wells or other means. As always in comparing any two areas, it is not possible to draw an exact parallel between the Grand Prairie and the Yazoo Delta because the geologic and hydrologic conditions in the two areas are not identical. Nevertheless, the points of similarity are numerous enough to emphasize the need for anticipating similar problems in the Delta, and for the State or local agencies to take appropriate action in spacing wells, regulating withdrawals if necessary, and practicing artificial recharge. To this end comprehensive ground-water studies were begun in the area late in 1953 as a cooperative State-Federal project, and are continuing today with the Mississippi Board of Water Commissioners as the cooperating State agency. These studies will provide a sound basis of hydrologic facts for use by the State and its administrative subdivisions in promoting full yet safe development of the large ground-water resources of the Delta.

Conclusion

Lessons to Be Learned from Problems

What are some of the lessons that might be gained from the case histories presented, and from the results of other hydrologic studies throughout the Nation? Certainly one of the first needs is to recognize that, although the emphasis here has been placed on ground water, the problems of ground water and surface water are inextricably bound together. It is impossible to consider use of the one resource without taking into account the extent to which the other may be affected. The effect may be adverse, as where pumping of ground water depletes the flow of a stream whose water is already fully utilized, or where new diversions from a stream reduce the recharge to a downstream aquifer. On the other hand, it may be possible to call on surface water to remedy a shortage in ground water, or vice versa.

In all six problem areas described, surface water is of significance in one respect or another. In the Roswell basin, surface water is involved because the water of the Pecos River is fully appropriated, and that stream cannot be counted on as a source of water to make up for a shortage in ground water. More than that, pumping and consumptive use of ground water deplete the river, whose base flow depends to an important extent on the upward seepage of water from the artesian basin.

In the High Plains also, surface water is insufficient to replace the diminishing resources of ground water. Projects that are proposed for construction would make water available from the Canadian River to meet a substantial part of the municipal and industrial needs of the area, but only a

fraction of the irrigation requirement. Some of the water that runs off into the many shallow ponds of the Plains also will be salvaged for direct use or for recharge to the aquifer through wells, but this also will not be a complete answer to the problem. Nevertheless, whatever surface water can be made available will contribute importantly to the economy of the High Plains.

In southeastern Florida the control of surface water is perhaps the most essential ingredient in the successful use of the abundant ground water. The area is one in which a delicate balance must be preserved between replenishment and disposal of water. The replenishment from rainfall is large, and often occurs at nearly unmanageable rates, as when a hurricane dumps many inches of water over the whole area in a day or two. On the other hand, the combination of a very highly permeable aquifer and a system of drainage canals results in rapid draining of the water during dry weather, unless steps are taken to halt it.

It would be possible to bring out the significance of surface water in the other three ground-water problem areas we have discussed, but to do so would only emphasize further the point we are making—that ground water and surface water must be considered together, both as sources of water and as interrelated resources neither of which can be developed safely without considering the effect on the other.

Now that we have established that point, we might mention another which hardly needs mentioning to this audience. That is the point that our national water supply is not diminishing, so far as we can ascertain, but is remaining about the same over the long term. There are wet and dry cycles ranging in length from a few years to a few decades that must be reckoned with, and less definite wet and dry trends which may have periods of 50 to 100 years or more. But we have no reason at this time to fear that our Nation or our planet is gradually drying up.

Thus it is comforting to reflect that we can count on the same total water supply—in the long run. But we still have to face the problems brought on by the droughts that recur periodically—like the one that has persisted in much of the Southwest ever since the record-breaking rains of 1941, and the one that has affected a sizable part of the Middle West and East during the last 2 or 3 years. Each new drought brings problems more severe than its predecessors, because each year we are using more and more water and are approaching the limits of supply more and more closely in ever more numerous localities—and when we take advantage of normal or wet spells to expand our water uses to the limit, we are compounding the troubles that will have to be faced when the inevitable drought comes. The water shortage in New York City in 1949 and 1950 provides a dramatic example of this kind of situation. There it did not even require much of a drought to bring home the impossibility of continuing year after year to take advantage of the extra water provided by a series of wet years. Fortunately, the water shortage caused relatively little real hardship, and the city already had plans for additional sources which would have been in service had it not been for the disrupting effect of World War II. Nevertheless, the situation provides an illuminating example of what may happen in a less spectacular but perhaps more serious way in other areas if development and conservation of water do not keep up with demand.

With our water supply relatively uniform yet reduced significantly for periods of one to several years by drought, and with our water demands rising every year, what can we do? Obviously, we will do what we must do. We will buy more water if we can afford to, we will waste less and reuse more if

we cannot afford to buy more, or we will simply draw in our horns and reduce our water demand, substituting activities or methods that require less water.

Basic Attack on Problems

To accomplish the overall purpose of getting the water we need at a price we can afford to pay—or, to put it somewhat differently, to keep our water demands within our ability to pay—we need to do several things:

First and foremost, we must find out how much water we are using and how much there is that we can use. In this simple statement are involved the not inconsiderable task of keeping an up-to-date inventory of our water uses, and the very much larger, almost overwhelming task of measuring the Nation's water supply. That huge job will require that we fill in the many large gaps in our present knowledge of the basic physical principles that control the occurrence of water in different environments and the forces involved in the movement of water from one environment to another. As both an essential part and an extension of this research, we must utilize the knowledge of basic principles to locate and characterize each of our surface and subsurface sources of water and to estimate closely the amount of water available from each under various situations of weather and water-use pattern.

Armed with this knowledge, we must take up the individual problems as they arise, anticipating them insofar as possible so as to enable solving them in their earliest stages while they are still easy and inexpensive to solve. We must recognize that each problem is different, as was stressed earlier; that each is a product of the local geologic and climatic environment and history of water development; that for most of the major problems the solution will be difficult, complicated, and expensive, and will require considerable public education to secure adequate public support; and that a solution which might work in one area may be impractical in another hydrologically similar area because traditions of water use that have become expressed in law may make it unworkable.

Obviously, the most efficient utilization of our water resources would require planning for the long term, and control of development. In this country legal control of the use of water lies in the States, and interstate problems are handled by means of compacts assented to by the Congress, or by means of interstate lawsuits decided in the Supreme Court. Because interstate difficulties form only a small part of our water problems as a whole, it is within the framework of State law that the planning and control of water use must be accomplished. Though occasionally existing law may handicap efforts to achieve long-range planning and adequate control of water development, and public sentiment may not favor the changes in the law that would be necessary for effective water use, in most States much more can be done along these lines than has been done in the past.

Of all the groups in our society, the engineers are among the best qualified by temperament, training, and experience to provide the contact between the research scientist and the legislator that will be essential to the enactment of water legislation. The American Society of Civil Engineers has one of the principal roles to play in this national effort; and I am happy to predict that, in no small part as a result of the Society's efforts, another century will see our Nation secure in its water supply upon which the national economy so largely depends.

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SURFACE WATER RESOURCES[1]

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SYNOPSIS

For any stream the average annual runoff over a period of years tends to conform to a characteristic geographic pattern, but year-to-year runoff varies widely. Annual runoff in the United States during the four-year period 1953-56 is analyzed briefly. The paper discusses the rapid expansion of irrigation in the eastern States, and emphasizes the importance of hydrologic analyses and legislation to meet problems that will arise as a result of competition with other uses.

INTRODUCTION

The Nation is becoming increasingly water conscious, largely because of water problems that have developed in many communities during recent years. The rapidly increasing use of water has caused some concern over the possibility that the future economic growth may be handicapped by an inadequacy of water. As a result of diminished precipitation, streamflow during the past several years has been generally below normal over much of the southern half of the United States. In the Southwest, drought has prevailed for several years, and elsewhere in the Nation many communities have experienced temporary water shortages. Although there appears to be no significant dwindling of the total water supply, the margin between supply and demand is narrowing.

Availability of water likely will be a limiting factor in the further expansion of certain uses of water in some areas, and it is almost inevitable that conflicts between types of uses will become more frequent. There is need for a nationwide coverage of up-to-date legislation governing the use of water.

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Distribution of Runoff

The total amount of water on the earth is almost limitless as compared with the needs of man, and as far as we know, it remains practically constant in amount. Through the natural phenomena known collectively as the hydrologic cycle, a small part of this water is distributed over the earth, and becomes available to man for his use. The hydrologic cycle is affected by many natural influences. As a result of some of these influences, such as physiography and prevailing winds, distribution of moisture tends to follow a geographic pattern. As a result of other and more obscure influences, the pattern of moisture distribution is irregular with respect to time.

A generalized geographic pattern of average annual runoff in the United States is shown in figure 1. This figure illustrates what we all know so well, that water is much more plentiful in the East than in the West. The average annual runoff is less than 1 inch over large parts of many of the Western States—an aggregate area of more than 25 percent of the total area of the United States. On the other hand, here in the Eastern States it is mostly in excess of 10 inches; in fact, in twelve States east of the Mississippi River it is 20 inches or more. The nationwide total averages approximately 9 inches per year or about 30 percent of the average annual rainfall.

Much more complex is the time distribution of moisture, and in view of the rapidly mounting use of water, the water shortages experienced in recent years have led to some concern over the possibility of a downward trend in water supplies as a result of long-term climatic trends. Streamflow records collected by the Geological Survey over a period of 50 years or more do not indicate a persistent downward trend, nationwide, when adjustments are made for man-made changes in the consumptive use of water. Far-reaching climatic trends appear to be so gradual as to be scarcely discernible within a lifetime. Of much greater immediate significance is the great year-to-year variability in weather and the apparent tendency for wet years and dry years to group together in sequences in which one or the other predominates.

The year-to-year variation in streamflow is due to a combination of climatic and hydrologic conditions. The variation in streamflow from year to year is greater than the variation in precipitation because of the fact that streamflow is a residual. With Nature's take remaining relatively constant from year to year, the percentage difference in runoff is much greater than the percentage difference in rainfall. Furthermore, and for the same reason, the percentage difference in runoff from year to year is greater in areas of low annual precipitation than in areas of high annual precipitation. Thus, streams in dry climates tend to vary between wider extremes, percentage-wise, than streams in the more humid climates. This difference in pattern of time distribution is of considerable significance to water users.

The effect of difference in streamflow variability is demonstrated by a comparison of the variation in annual discharge of Neosho River near Parsons, Kansas, and Pearl River at Jackson, Mississippi, for the 27-year period 1929-55. For the Neosho River where the annual runoff is about 5 inches, the annual discharge ranged from 9 percent to 410 percent of the median annual discharge, while for the Pearl River where the annual runoff is about 20 inches, the range was only 42 percent to 222 percent.

For purposes of general classification of runoff conditions, the Water Resources Review, a monthly press release by the U. S. Geological Survey, classifies streamflow as deficient, normal, or excessive, and also expresses

the discharge as a percentage of the median for the base period 1921-45. Deficient annual discharge is defined as the discharge among the lowest 25 percent of record for the month. Applying this definition to the annual discharge of the Pearl and Neosho Rivers for the actual period of record, we find that the deficient range in annual discharge on the Neosho River is the discharge below about 30 percent of the median annual discharge, and on the Pearl River, below about 70 percent. Thus, on the Pearl River, an annual discharge of 65 percent of the median would be considered deficient, while on the Neosho River an annual discharge of 35 percent of the median would be within the normal range. These two records were picked because they are streams in this general area that illustrate the point desired to make, but are not by any means extreme examples of the runoff conditions in a humid area versus those in an arid area.

Droughts

What is a drought? Many general definitions can be found. Webster's New International dictionary defines drought as "such dryness of weather or climate as affects the earth, and prevents the growth of plants," which probably expresses a widely-accepted concept of drought, at least from a qualitative point of view. Thus far, no satisfactory formula has been devised that defines a drought quantitatively in terms of rainfall, streamflow, or soil moisture.

Droughts may be classified broadly as (1) water supply droughts and (2) agricultural droughts. Both may be the result of climatic conditions, or may be the result of the demand outgrowing the supply. So long as needs for water in a community are satisfied there is no evidence of droughts, however small the supply may be. Conversely, when needs for water exceed the supply, an apparent drought exists even though the deficiency may be no greater than can be expected to recur every few years. But when deficiencies occur simply because the supply has been over-developed, can such deficiencies correctly be termed droughts?

The effect of a dry period varies widely with the type of vegetation and with the type of irrigation common to an area. Deep-rooted grasses and trees can withstand a drought of several weeks during which shallow-rooted vegetation will be badly damaged. In areas where long periods of no rain occur every year native vegetation adapts itself to the climate and man makes provision to irrigate his crops. In these areas droughts occur only when the supply of water is insufficient for irrigating the usual amount of acreage.

In a country as large as the United States and subjected to such a diversity of climatic influences, it is only in an unusual year that streamflow is within the normal range in all parts of the country. The occurrence of moisture deficiencies and consequent deficient runoff in some parts of the country may be considered normal. However, the present drought in the Southwest is of such wide areal extent and of such persistence that it undoubtedly is one of the major droughts in our history.

For the past several years the Geological Survey has been in the process of preparing a report on droughts in the Southwest. The Survey has been waiting for the drought to be over before completing this report but has now decided to close the report with the year ending Sept. 30, 1956, even if this makes it an interim rather than a final report. More than one wet year will be needed before anyone can say that the Southwest drought is really over.

The Southwest drought began in Arizona and New Mexico about in 1943, and except for occasional temporary relief, has persisted. It gradually spread eastward and northeastward as far as Illinois. However, runoff for the water year ending Sept. 30, 1952, was normal or above in all the United States except the Gulf States (although during the summer, runoff was deficient over large areas). Hence, for the purpose of this paper, attention is directed mainly to annual runoff for the four water years 1953-56.

Annual runoff was deficient or in the low normal range over most of the southern United States, from the Atlantic to the Pacific, during the four years 1953-56. The most notable exception was an area embracing the States of Mississippi and Louisiana, and parts of the adjacent States, where runoff was above median—even excessive—in 1953. In 1954 the areas of deficiency spread widely. Conditions improved somewhat in 1955, and even in the Southwest the drought pattern showed signs of disintegrating. The year 1956, however, gave no definite evidence that the drought was generally abating; in fact, annual runoff reached new record lows at three key gaging stations in Arizona, and one in Texas, as well as two in Florida, one in Virginia, and two in western North Carolina. These key stations are used by the Survey as indices of runoff conditions in the general area where they are located. Annual runoff in the State of Oklahoma as a whole for the calendar year 1956 was the lowest of any calendar year since collection of streamflow records was begun. The power output of Bagnell Dam in Missouri in 1956 was the lowest in 25 years of record, and was only 12 percent of the average annual output for that period. Combined storage in Elephant Butte and Caballo Reservoirs in New Mexico at the end of the year 1956 was only 5 percent of the 18-year average, and only a little more than 5 percent of the normal annual irrigation demands. These few statistics show that drought conditions are widespread and most serious.

The number of deficient years in the 4-year period 1953-56 serves as an index of the deficiency of annual runoff in the United States during that period. The number of years of deficient runoff at each index station used in the Water Resources Review is shown on figure 2.

Deficient runoff gas previously has been defined as the runoff during the lowest 25 percent of the years in the base period 1921-45. Accordingly, a deficiency in annual runoff during one year of the four could be expected on the average. Moreover, owing to the fact that wet and dry years do not follow in regular sequence, it could be considered normal to find a scattering of points of no year, one year, or two years deficient out of four, but only in an area of protracted drought could a consistent pattern of two to four deficient years out of four be expected. The lines joining the points of equal number of years of deficiency emphasizes the severity and scope of the drought.

The area of the drought during the last four years is delineated by a consistent pattern of "four's," changing to "three's" and "two's" around the outer boundary. One striking feature of the drought pattern is the tongue that protrudes across Missouri and into Illinois. Except for an area in the Carolinas, no persistent pattern of deficiency of annual streamflow is indicated in the southeastern and mid-Atlantic States, although the predominance of "two's" indicates a greater tendency towards deficient runoff than would be expected, on the average, in a four-year period.

This presentation focuses attention on annual instead of monthly or seasonal runoff. Although seasonal deficiencies, such as the drought in northern Georgia in 1954, may thus be obscured, it is believed that a persistent

deficiency in streamflow is better illustrated by annual runoff than by the much more erratic pattern that would be indicated by monthly or seasonal runoff. This is not to imply that water shortages of relatively short duration are any less serious to a particular area than those based on annual runoff; in fact, just the reverse could be the case, but for the purpose of this discussion the annual runoffs are believed to be more meaningful.

Development of Water Supplies

Deficiencies as well as floods can be considered as normal events in that they will continue to occur. Water needs will have to be reconciled with the supply as distributed by Nature. Through modification in water needs and utilization practices, and through greater expansion of facilities for storage and conveyance it is possible to stretch the present supplies.

In some of the problem areas of the West, water supplies are already heavily taxed and further growth of the communities dependent on those supplies must involve some modification of uses of water. Some of the irrigated land in the West may have to be retired in order to provide water for use with a higher cash value. Undoubtedly, much can still be accomplished towards reducing waste of water by consumers. Also, more can be done toward elimination and control of non-beneficial water-loving plants, although the economic value of the water thus salvaged may not everywhere justify the expense involved. In some areas importation of water may be the answer, provided a source of supply is available and the expense of conveyance facilities is economically justified.

In the eastern United States the picture is generally brighter as far as availability of surface water is concerned. The East has had an increasing number of problem areas in recent years, but these areas have largely been industrial or population centers. Additional water usually is available provided it is feasible to reach out to more distant sources. In the East, more so than in the West, one answer is more storage reservoirs.

Supplemental irrigation is becoming increasingly popular in the East. According to the 1956 Directory and Buyers Guide, Irrigation Engineering and Maintenance, irrigated acreage increased significantly during the 5-year period ending 1955. States showing increases of more than 1,000 percent were Alabama, Mississippi, North Carolina, Kentucky, Tennessee, West Virginia, and Delaware. These percentage increases may be somewhat misleading if viewed in the light of the fact that the total irrigated acreage in the East is still less than 10 percent of the irrigated acreage in the 17 Western States, but the fact remains that the irrigation business in the East is booming. Conceivably, the ratio of irrigated acreage in the eastern United States to that in the West could be changed greatly in the next decade.

The greatest potential for further expansion of irrigation undoubtedly is in the eastern United States, where experience has proved that crop yield can be substantially increased by supplemental irrigation. That fact stands even though crops can be grown in the East with only the rain that falls during the growing season. In the future, a large share of the burden of increasing the Nation's food production may fall on Eastern agriculture, with a result that irrigated acreage may be increased many fold. With respect to irrigation, the East has several advantages over the West: base flow of streams is generally greater and more reliable; through supplemental, rather than almost

total, irrigation, a given quantity of water can serve a much larger acreage; objectionable salts, where deposited in the soil by irrigation water in humid areas, is at least partly leached out by rains, whereas in the West the leaching process must be accomplished largely by the application of irrigation water sometimes greatly in excess of crop requirements.

One of the primary requirements for irrigation from surfacewater sources is storage, but here again the East has its advantages over the West. Because of the smaller irrigation requirements and the greater stability of the streams, smaller storage projects will suffice. The criterion for design commonly will be, not how much water is available, but how much water is needed to tide the user over the deficient season. In the West the criterion for design, especially for the larger irrigation projects, commonly is how much water is available. Some of the larger reservoirs in the West store a comparatively large part of a normal year's runoff; more water could be used were it available.

A large expansion of irrigation in eastern United States will, in some lolocalities, lead to conflicts with other uses, particularly where irrigation results in the depletion of streams serving industrial and population centers. Irrigation use is consumptive, and a certain degree of depletion will occur, possibly to the extent of materially changing the low-flow regimen of some streams. Industry is well established in the northeastern States, but in the southeastern States, both industry and irrigation are expanding rapidly, for nowhere in the United States are water supplies so generally favorable for further development as in the southeastern States, with the possible exception of the more humid parts of the Pacific Northwest. Although irrigation use is consumptive, it may have little effect on water quality except where return flow carries salts that have been leached out of the soil to which irrigation was applied. This is not believed to be a matter of great concern where irrigation is supplemental in character and where streamflows are rather large compared to this type of use. Industrial use, although essentially not consumptive, almost inevitably results in some pollution, especially when a stream is used as a means of disposal of industrial wastes. Hence, conflicts involving both quantity and quality of water may be expected where industry and irrigation compete for the use of the same stream.

Need for Legislation

A real need exists for studies to anticipate these conflicts--to delineate the problem areas and to determine the probable nature of the problems that may develop. Studies should include collection of streamflow records where adequate records do not exist, and analyses of these records including the effect the various uses will have on stream regimens. Studies of the surface-water resources of an area are, of course, incomplete without similar studies of the ground-water resources, as in many areas the two are closely interrelated. For example, extensive use of ground water of an area may have a pronounced effect on the base flow of the streams in the area. As to degree and timing of this effect, only detailed studies will tell the story. Furthermore, the effect of diversions and uses on the quality of these waters is in certain cases of equal importance to the effect upon quantities available.

In many States where water laws do not exist or are not conducive to a logical development of the water resources, much thought is being given to

developing laws governing the use of water. The developing of water laws that are practicable, workable, and equitable requires an understanding of the engineering and hydrologic aspects of the problems the laws are designed to meet. Much thought is being given to this important matter right here in the South. The State of Mississippi is an example. It is much better to be prepared with an ounce of prevention than to have to worry with the pound of cure. Even here in the humid areas water can no longer be taken for granted, and assumed to be available for all in an inexhaustible supply.

Members of the American Society of Civil Engineers undoubtedly will have a prominent role in studying the future needs for water in all parts of the country, and in developing wise laws to regulate its use for the maximum benefit of all. The legal profession can phrase the laws so that they will stand the test of the courts, but the engineering profession must furnish the facts and the interpretation of those facts so that such laws will conform to the laws of Nature, and only thereby will they be practicable, workable, and equitable.

CONCLUSIONS

The Nation's total water supplies probably will not be substantially greater or less in the future than in the past. Deficiencies of water such as have been experienced in recent years are recurrent events, and must be taken into consideration in planning for the future. In view of a rapidly growing population and a rapidly increasing use of water, it is important that the hydrologic characteristics of the streams and other sources of water be understood, and that legislation be directed towards equitable distribution of water to meet even greater requirements than now exist.

ACKNOWLEDGMENTS

Substantial advice and assistance in preparing this paper was received from engineers of the hydrologic studies unit, especially from Conrad D. Bue, Assoc. M. ASCE.

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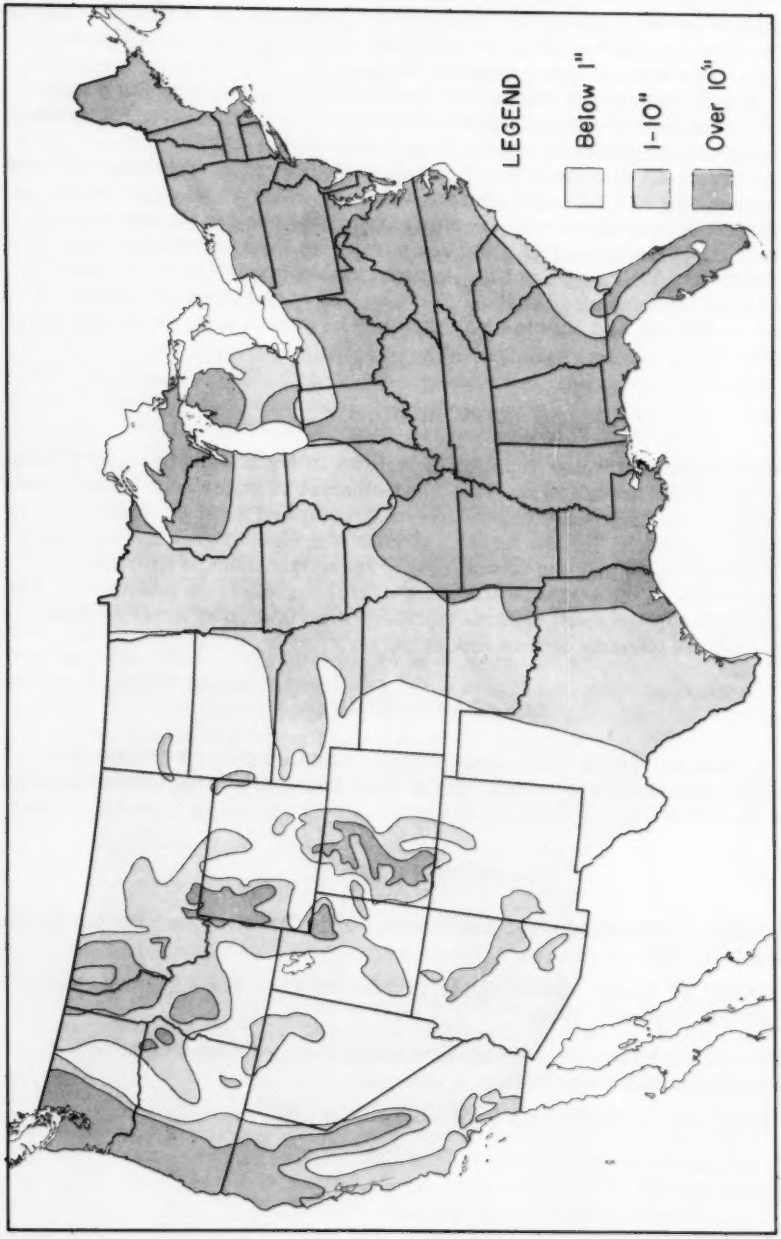

Figure 1.--Average annual runoff.

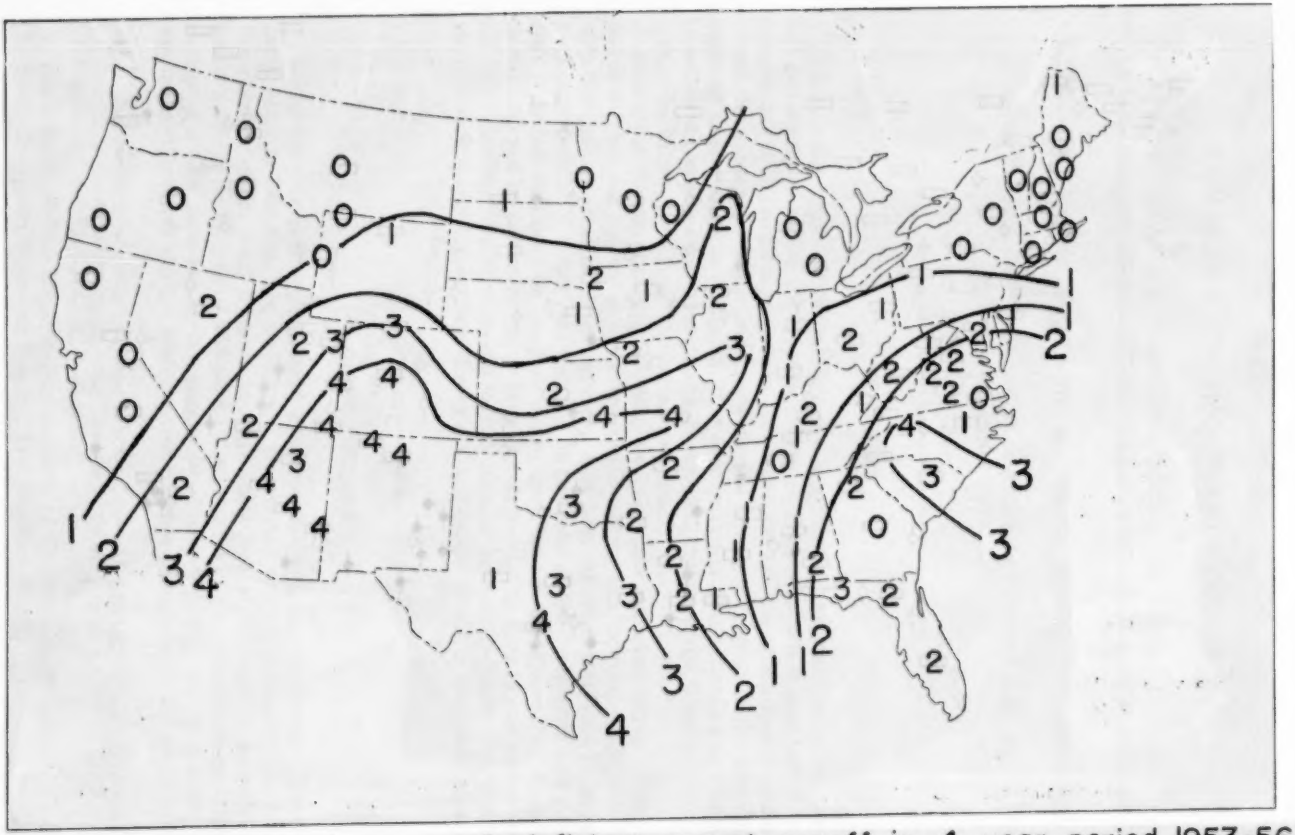

Figure 2.-- Number of years of deficient annual runoff in 4-year period 1953-56

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HYDRAULIC MODEL STUDY OF HYPERION SEWER INTERCHANGE

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(Proc. Paper 1273)

SYNOPSIS

A 1:20 scale model of the interchange structure at the junction of two outfall sewers approaching the Hyperion Sewage Treatment Plant was tested in accordance with the Froude law, to determine the effects of various flow distributions among the two influent sewers and channels leading to the new and existing headworks. In particular, the configuration of the throat was studied to ascertain whether the interchange could pass the required flows without unduly backing up either of the outfall sewers.

Maximum storm, peak dry weather, average, and minimum flows were tested for normal operating condition and a variety of abnormal conditions with one unit or another out of service. The best throat design was found to be a combination of a long-pointed tail on the pier at the confluence and a 1-ft radius cylindrical nose on the pier facing upstream at the diversion. Energy is transferred by turbulent shear from the fast-moving flow in one sewer to the slow-moving flow through the throat, so that the energy grade line downstream is actually higher than that upstream of the throat in the other sewer.

INTRODUCTION

Under the proposed enlargement program of the Hyperion Sewage Treatment Plant at Los Angeles, California, provisions are being made to increase the capacity of the plant from an average daily flow of 245 mgd to an anticipated average daily flow of 420 mgd. The present plant utilizes the high rate activated sludge process for secondary treatment (see Fig. 1). It is now served by the North Outfall Sewer which carries sewage contributed by inland

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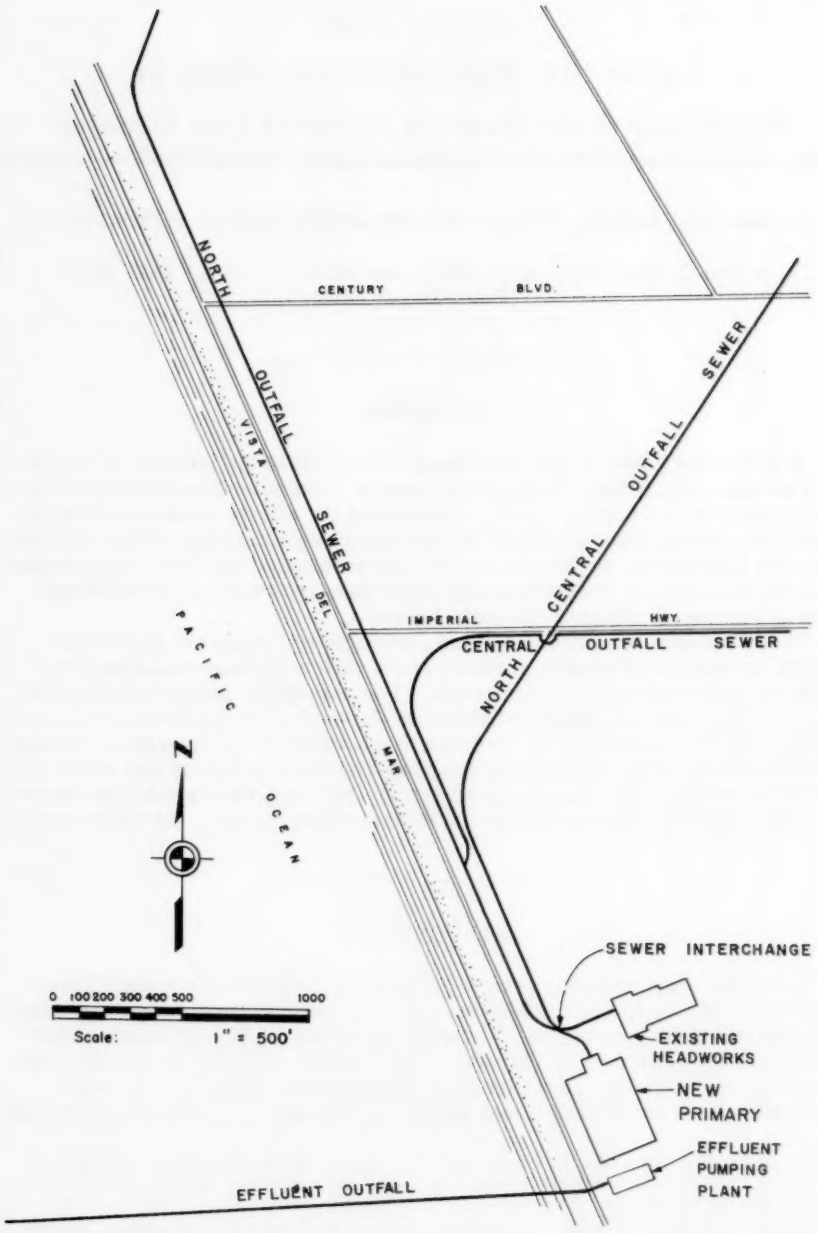

FIG. 1 LOCATION MAP OF SEWERS TO HYPERION TREATMENT PLANT

as well as coastal areas. The composite flow arriving at the plant contains slightly more salinity than is considered desirable for potential reclamation purposes. A new trunk sewer, designated the North Central Outfall, will supplement the existing North Outfall Sewer. Sewage in the North Central Outfall Sewer is expected to have no objectionable salinity quality.

Limitations of space, requirements for conformation to existing construction and other restrictions have made it necessary to provide an interchange of flows between the two trunk sewers immediately upstream of the existing headworks (see Fig. 2). This interchange is to permit the entire flow of low salinity sewage from the North Central Outfall to enter the existing primary tanks. The additional flow required for the proper proportion, (see Fig. 3) will be bled from the North Outfall Sewer. Thus, the saline content of this combined flow will be much less than that at present and the resulting effluent could be reclaimed. A maximum of the flow in the North Outfall Sewer will then be given primary treatment in the new headworks and settling tanks and thence be discharged several miles to sea.

An elaborate schedule of capacity requirements for the interchange structure has been drawn up (see Fig. 3), specifying the flows in the outfall sewers and the proper distribution of these flows to the influent channels. Of the four conditions shown, condition A for normal operation was the criterion for the preliminary design of the interchange shown in Fig. 2. Condition B for maximum quantity of secondary effluent will be the operating condition when the existing secondary tanks are at maximum capacity. Sewage in the New Primary will be treated and discharged directly to sea. Conditions C and D were to be studied from the viewpoint of emergency operation during temporary shutdown of part of the plant. In addition to these flow requirements, the following specifications were to be met as closely as possible:

1. In order to minimize salinity in the sewage receiving secondary treatment, the flow to the new primary should be exclusively from the North Outfall Sewer, except for abnormal condition C, which calls for backflow through the throat.
2. For normal operation the North Outfall Sewer should not be backed up to a hydraulic grade line at point 1 (see Fig. 2) greater than 36.0 ft.
3. The water surface in the interchange structure should be as smooth as possible, to minimize release of hydrogen sulfide gas.
4. Sedimentation in the interchange should be held to a minimum.

Hydraulic Model Theory

Because raw sewage, such as that flowing to the Hyperion Treatment Plant is essentially water, the use of a hydraulic model is a reliable method of attacking sewer design problems which are not amenable to theoretical solutions. To state the case for hydraulic models briefly, it is possible to establish simple dynamic similarity between a scale model and its prototype if it can be demonstrated that one type of force, e.g. gravity, viscous shear, surface tension, etc. predominates over the other types in its relation to the inertia forces which cause (or resist) motion in the fluid. In most cases of liquids with a free surface, involving flow around obstructions or through constrictions (as in the case of the throat of the interchange structure), the predominant force is gravity and the Froude model law will apply[3]. Inasmuch as

3. Fluid Mechanics, by R. L. Daugherty and A. C. Ingersoll, Chapter 6, McGraw-Hill Book Co., New York, 1954.

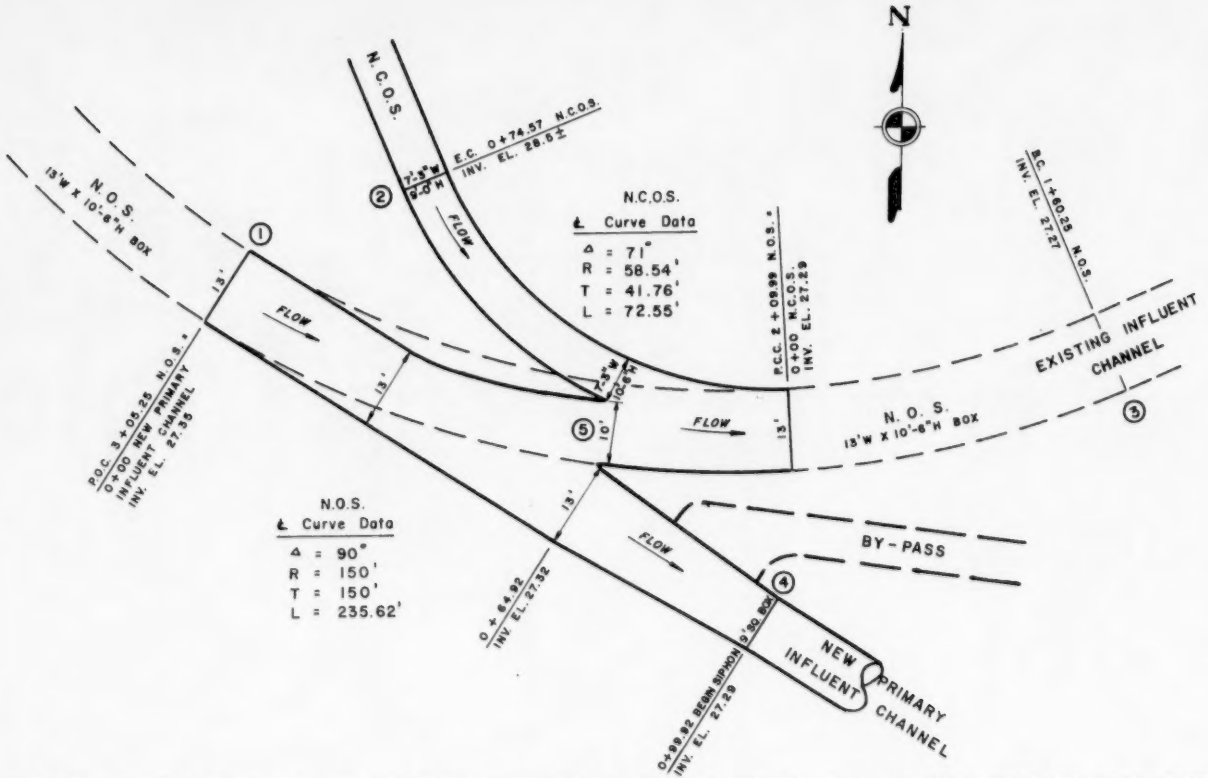

FIG. 2 PRELIMINARY DESIGN OF INTERCHANGE (ON WHICH MODEL WAS BASED) AND IDENTIFICATION OF REFERENCE POINTS.

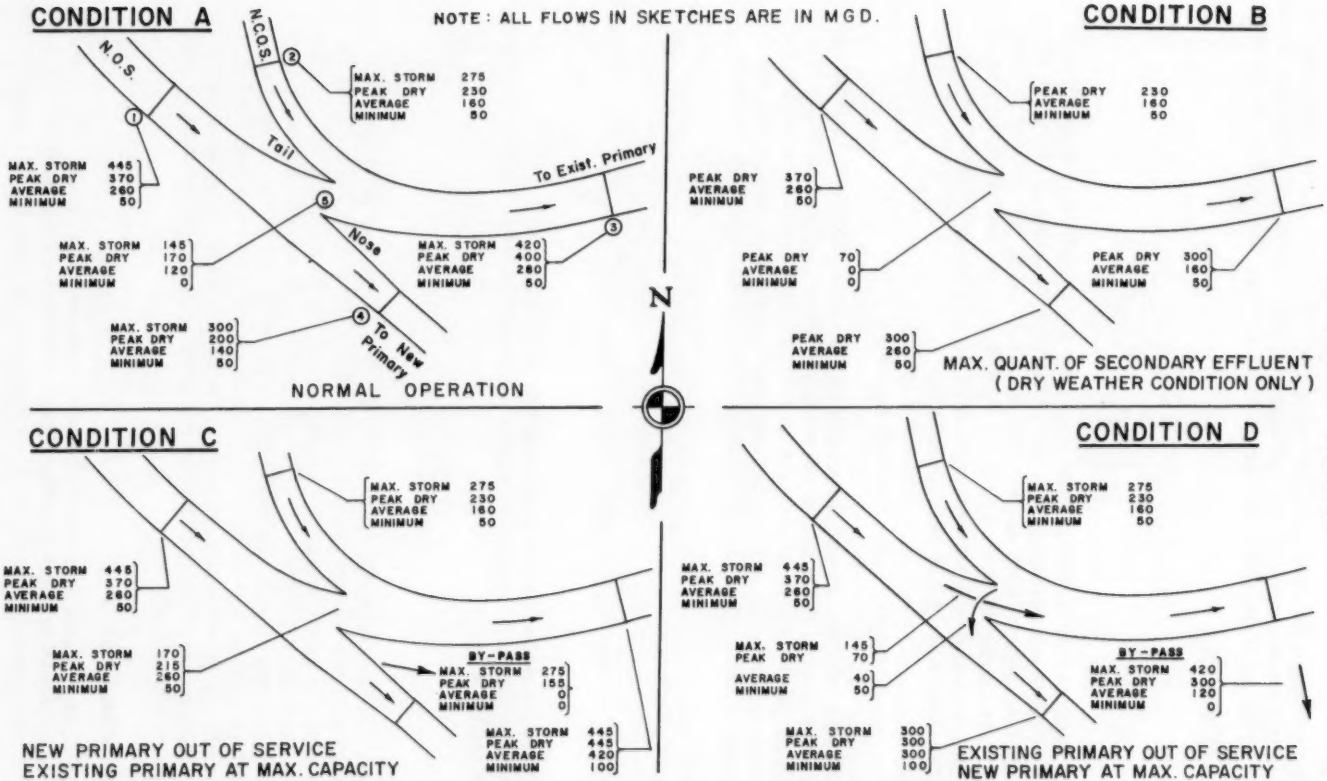

FIG. 3 DESIGN CRITERIA FOR INTERCHANGE CAPACITIES

all of the sewage flow here considered is carried to, in, and from the interchange in open channels, the Froude law was indicated for the Hyperion model study. That is, if the model is built to scale and operated so that the Froude number,

$$N_F = \frac{V_p}{\sqrt{L_p g}} = \frac{V_m}{\sqrt{L_m g}} \tag{1}$$

is the same in both model and prototype, all linear hydraulic quantities, such as water depth and velocity head, are in direct proportion as the scale ratio, and other quantities, such as velocity, discharge, etc., are related through Eq. (1) and its derivatives. In Eq. (1) V_p and V_m are representative velocities in prototype and model respectively, while L_p and L_m are representative lengths. The model scale ratio is defined,

$$L_r = L_m / L_p \tag{2}$$

The Froude number, like all similarity parameters, is seen to be a dimensionless quantity. If the model and prototype were operated at different values of the gravitational constant, g, it would be necessary to include the different values in Eq. (1), but for ordinary hydraulic models, g may be assumed constant. Eqs (1) and (2) then establish the important relations,

$$V_m / V_p = \sqrt{L_m / L_p} = \sqrt{L_r} \tag{3}$$

$$Q_m / Q_p = V_m L_m^2 / V_p L_p^2 = (L_m / L_p)^{5/2} = L_r^{5/2} \tag{4}$$

which are the ones utilized in this study.

To establish that the Froude model law is applicable it must be shown that the Reynolds number,

$$N_R = \frac{LV}{\nu} \tag{5}$$

(where ν is the kinematic viscosity of the fluid) indicates non-viscous or turbulent flow in both model and prototype. For a Froude model utilizing the same liquid in model and prototype, N_R is seen to vary as $L_r^{3/2}$. The lower limit on the allowable scale of a Froude model is generally established as that at which the flow in the model is just securely out of the laminar range.

Surface tension forces enter the Froude model picture in that the liquid surface in the model would not have the same tendency to break up as would the surface of the prototype, but the effect on other quantities such as depth of flow should be negligible.

Choice of Model Scale

The first problem in any model study is the choice of scale. The lower limit is governed by the Reynolds number criterion mentioned above, and the upper limit is set by economics. The choice of 1:20 scale for this model yields

the following values of Reynolds number (with the representative length set at the customary four times the hydraulic radius) at minimum flow in the North Outfall Sewer:

(width = 13.0 ft., depth = 7.32 ft., velocity = 0.82 ft / sec, $\nu = 0.00001$ ft^2/sec.):

$$\text{Prototype: } R = \frac{7.32 \times 13.0}{1.30 + 2(7.32)} = 3.44 \text{ ft}$$

$$N_R = \frac{4(3.44) \times 0.82}{0.00001} = 1.12 \times 10^6$$

$$\text{Model: } N_R = \frac{1{,}120{,}000}{20^{3/2}} = 12{,}540$$

The model value is about 5 times larger than the critical value for commencement of laminar flow. For the conditions of most interest, at peak and maximum storm flows, these values of Reynolds number will be multiplied by about 7 or 8, showing that the effect or viscous forces is even more remote. As the 1:20 scale was considered to be the maximum economically feasible, it was fixed at the outset and there was no cause for later reconsidering this decision.

Construction of the Model

It was decided to model not only the interchange structure alone (as shown in Fig. 2) but approach sections of the North and North Central Outfall Sewers as well as the channels leading to the existing and new headworks. The principal features of the model layout are shown in Fig. 4.

Construction Details

The base of the model was of 3/4-inch plywood, mounted on a framework of 2 x 4-inch joists. The inlet boxes were made of 1/2-inch plywood and sealed with caulking compound. The channel walls were made of 1/4-inch plywood, oriented so that the short-radius bends in the wall were with the grain on two of the three layers, and glued with waterproof glue to supporting strips nailed to the base at the outside of the channel wall. This procedure proved most effective in providing smooth walls with no leakage. The transition piece in the line to the new headworks, was formed of 16-gauge steel plate and welded to the 5-inch pipe.

The key to the design of the interchange structure lies in the details of the "tail" at the confluence of the North Central Outfall and North Outfall Sewers, and of the "nose" at the diversion of the flow to the new primary influent channel from the North Outfall Sewer. For this reason, the tail and nose pieces were made so that they could be changed easily. The various designs tried out are shown in Fig. 5.

Water was circulated by means of two low-head pumps, each having a capacity of about 0.4 cfs, more than adequate to meet the maximum flows in the North and North Central Outfall Sewers.

Discharge Measurement

The North Central Outfall flow, Q_2, was supplied from a constant head tank

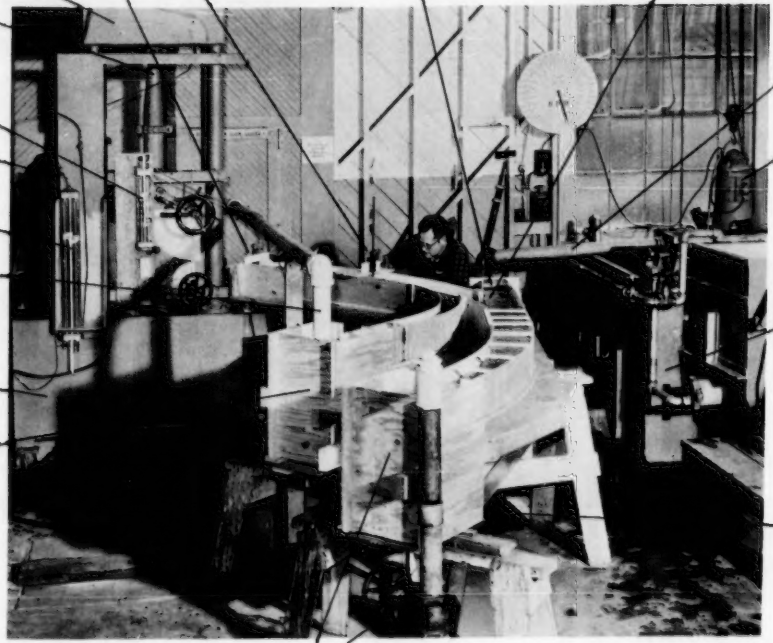
Constant head reservoir for North Central Outfall supply
To existing headworks (Q_3)
To new headworks (Q_4)
Point gage
Interchange structure
Venturi meter for North Central Outfall supply
Manometer for N.O.S.
Delivery pipe for N.O.S.
Pump motor for N.O.S.
Pump motor for N.C.O.S.
Manometer for N.C.O.S.
Delivery pipe for N.C.O.S.
Supplementary reservoir for N.O.S. intake
Return reservoir
Inlet box for North Central Outfall Sewer (Q_2)
Venturi meter for N.O.S. supply (underneath model table)
Inlet box and control valve for North Outfall Sewer (Q_1)

Fig. 4. General view of interchange structure model

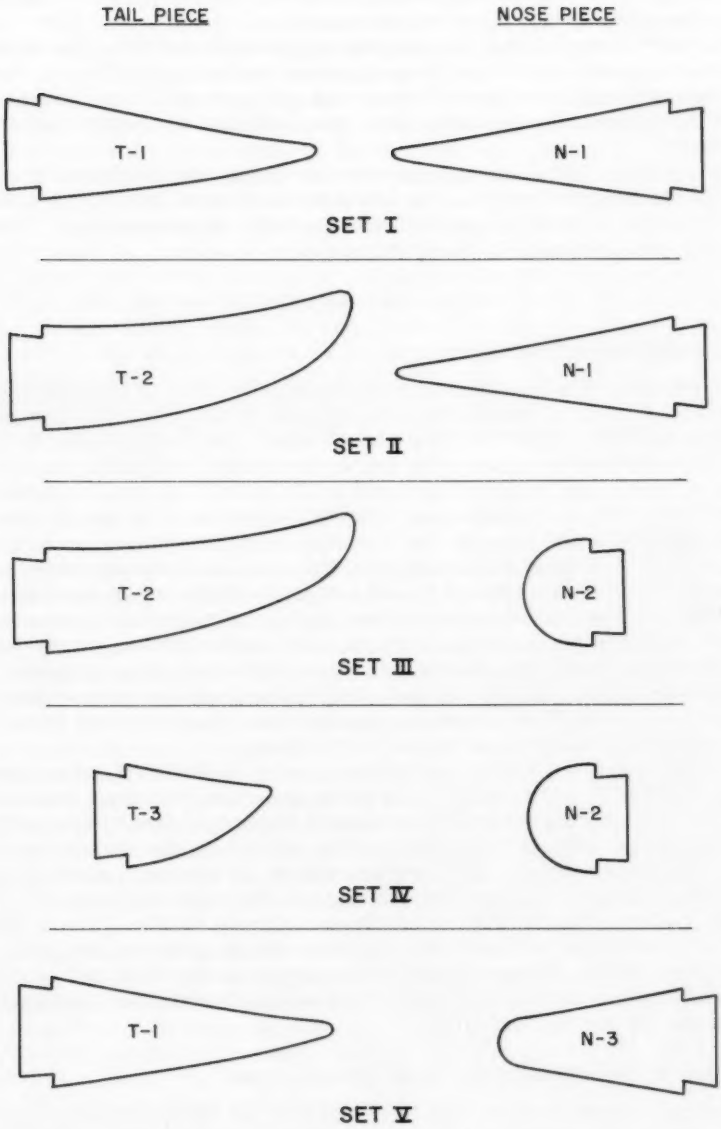

FIG. 5 INTERCHANGE THROAT DESIGNS TESTED

with a skimming weir at about 12 ft above the floor level. The supply was metered by a 2-1/2 inch venturi throat located at the outlet of the reservoir and connected to an air-water manometer.

The North Outfall Sewer flow, Q_1, was supplied directly from the other pump which drew from the auxiliary reservoir to the right in Fig. 4. The flow was measured by a 3 x 1-5/8 inch venturi meter which was installed in the delivery line under the model table, and connected to a water-mercury manometer.

The discharge to the new primary headworks, Q_4, was measured by another 3 x 1-5/8 inch venturi meter. The entrance condition to the existing primary headworks was modeled simply with an adjustable rectangular weir. The flow, Q_3, was obtained from the relation,

$$Q_3 = Q_1 + Q_2 - Q_4 \qquad (6)$$

Establishing Slope of the Model

The bed slope of a Froude model represents the effect of frictional forces and as such it must be established independently of the prototype slope and Froude model law. Where the reach of a channel constriction under study is short it is common to employ a flat bed on the model, recognizing that the frictional forces will be small compared to the gravity forces occasioned by differences in hydraulic grade line. For longer reaches it is usually attempted to establish the bed slope so that the depth is in proper scale at all stations, with the hydraulic grade line being the same in model and prototype.

The slope of a model channel is ordinarily established by computing or measuring the slope required to produce depths corresponding to known or assumed depths in the prototype under various flow conditions. In the case of the Hyperion sewer interchange there were fortunately data, computed from measurements, showing the measured hydraulic grade lines at Stations 1 + 60.25 and 3 + 05.25 in the existing North Outfall Sewer, for two flows near the projected maximum storm and peak dry flows.

The first construction then, was of the existing North Outfall Sewer, extending roughly from Station 1 + 22.38 to the end of the 90° curve, Station 3 + 95.87. Then modeled values of the measured discharge rates of 420 mgd and 320 mgd were put through the sewer and the slope of the flat plywood bed was adjusted until the required depths were produced. It may be remarked that these flows represent accelerating sub-critical (M_2-type) surface profiles of nonuniform flow. The Froude number for a depth of 8.45 ft is 0.36. It was found that a bed slope of 0.0008 (as compared with the prototype bed slope of 0.0006) produced the required depths within approximately 0.02 feet prototype scale, and this was fixed as the slope which would eliminate the frictional forces from further consideration.

Discrepancies from True Model-Prototype Relations

Following this preliminary test, the section of the North Outfall Sewer at the interchange was removed and the interchange structure was installed as per the original design of Hyperion Engineers. Unfortunately this reconstruction process caused the bed to shift so that the invert at point 3 became 1/4 in. too high and the invert at point 4 became 1/8 in. too low. As it was not possible at the time these discrepancies were discovered to rebuild the bed and rerun the tests, the data have been adjusted to give the best prediction of

prototype behavior as obtained from the model study. These adjustments are explained in the section on test results below.

The design called for a transition section in the new primary influent channel from a 9-ft square section to a 9-ft diameter circular section, with a drop in the invert of 6.29 ft in the 12-ft long transition. In the model, the circular section was made to conform to a convenient 5-inch pipe size (prototype 8.33 ft). This did not affect the hydraulics in the open channel upstream, however, as it was necessary in any event to throttle this flow with a gate valve downstream from the venturi meter.

The design of the North Central Outfall Sewer called for a rise in the invert of approximately 0.8 ft in the curve just upstream of the junction. If properly modeled, this would have meant a gradual rise in the floor of the model channel on the curve of about 1/2 in. Inasmuch as this design was not yet firmly fixed, and the losses in the North Central Outfall Sewer were not critical, it was not felt worth the extra effort to put in this sloping floor, and so the bed of this channel was at the same slope as the rest of the model. The effect of this was simply that the energy grade line in the prototype at point 2 would be slightly higher than its counterpart on the model, for the same values of discharge and hydraulic grade line. This matter is considered in detail in the section on test results.

Test Procedure

A test consisted of establishing the required flow distribution for one of the conditions and flows shown in Fig. 3, e.g. condition A (normal operation) peak dry flow, and then determining the water surface elevation at the four locations of interest, points 1, 2, 3, and 4, identified in Fig. 2.

The normal procedure was to set the required inflows in the North and North Central Outfall Sewers, and then to adjust the gate valve to the new headworks simultaneously with the adjustable weir to the existing headworks so as to establish the proper value of Q_4, (and thus also of Q_3). To establish a reference value for water surface elevation, the hydraulic grade line at point 3 was made sufficient to pass the required Q_3 through the existing headworks, according to plant data supplied. This reference value was 35.72 ft for the maximum measured storm flow of 429 mgd. As this applied to flow through all detritors and all bar screens, however, it was also used for flows as low as 400 mgd. For lower values of Q_3, the HGL at point 3 was adjusted so that the head on the detritor weirs (set at elev. 34.00) would vary as the 2/3 power of the discharge.

As explained above, the slope of the base table shifted after construction with the result that the values of HGL at point 3 are some 0.4 ft higher than intended. This difference may conveniently be represented as an extra allowable loss across the bar screens. In other words, for values of HGL at points 1 and 2 reported in Table I of the results, there is an allowable loss of about 0.4 ft (at the high flows) across screens or other obstructions at entrance to the existing headworks. If no obstruction is present, then the water surfaces at points 1 and 2 will be lower by approximately this amount.

For certain runs involving low flow to the existing headworks and high flow to the new headworks, e.g. conditions B and D, it was necessary to throttle Q_3 in order to build up sufficient head to force the increased Q_4 through the new headworks. This head was estimated on the basis of assumed losses in the venturi meter, butterfly valve, bends and transitions in the new primary influent channel.

Table I. Summary of Model Tests

Notes: a) All data shown apply to throat combination V, with tailpiece T-1 and nosepiece N-3.

b) All discharge rates, velocities, and elevations are projected to prototype values.

Operating Condition	Flow Type	Discharge Rates, mgd.				Hydraulic Grade Line, ft.				Remarks
		NOS Q_1	NCOS Q_2	Exist Prim Q_3	New Prim Q_4	NOS (1)	NCOS (2)	Exist Prim (3)	New Prim (4)	
						27.35	27.37	27.27	27.30	Invert elev. as per prototype
A	Max. storm	445	275	420	300	35.87	36.23	36.16	35.94	
	Peak dry	370	230	400	200	36.09	36.29	36.15	36.25	Illustrated in Fig. 6
	Average	260	160	280	140	35.87	35.93	35.89	35.95	Illustrated in Fig. 7
	Minimum	50	50	50	50	34.67	34.67	34.72	34.65	
B	Peak dry	370	230	300	300	35.65	35.87	35.94	35.48	
	Average	260	160	160	260	35.57	35.67	35.81	35.33	Illustrated in Figs. 8 and 9: must throttle Q_3: 0.3-ft pocket S. side of nose
C	Max. storm	445	275	445	275	35.97	36.29	36.17	36.16	
	Peak dry	370	230	445	155	36.31	36.45	36.17	36.54	0.2-ft pocket on N. side of nose
	Average	260	160	420	0	36.55	36.45	36.16	36.71	Figs. 10 and 11 (max. flow thru throat) 0.4-ft pocket N. side
	Minimum	50	50	100	0	34.91	34.87	34.90	36.91	
D	Average	260	160	120	300	36.03	36.25	36.38	35.48	Must throttle Q_3: 0.4-ft pocket S. side
	Minimum	50	50	0	100	34.19	34.13	34.22	34.15	

Test Results

Summary data of all tests are included in Table I, which gives the prototype values for rates of discharge and hydraulic grade lines which may be expected. Table II presents detailed computations for energy grade lines and the head losses between points 1 and 3, points 2 and 3, and points 1 and 4. Visual observations are shown in Figs. 6 through 9, which indicate certain interesting aspects of the investigation.

Adjustments to Data in Tables I and II

The data presented in the tables have been adjusted in several ways to represent, with three exceptions, the best prediction of prototype behavior as obtained from the model study. It has been remarked in the section on construction of the model that at points 3 and 4 the model as actually tested had bed elevations different from the proper values. The amount of this difference was 0.38 ft prototype scale (high) at point 3 and 0.21 ft (low) at point 4. As these differences cause velocities at these points which are within 4 per cent of the proper values, the average velocities between upstream and downstream points are within 2 per cent of their proper values, or, in general, average velocity heads are within 4 per cent of their proper values. Head losses, being proportional to velocity heads, should therefore also be correct within about 4 per cent. As the numerical value of head loss in almost every case is less than 0.25 ft, however, it is not reasonable to consider head loss corrections to data showing losses to hundredths of a foot.

The foregoing explanation serves as the basis for the adjustment of model data, namely that head losses actually measured on the model were assumed to apply to equivalent tests on a hypothetical model - one which had a bed slope of 0.0008 as determined by the original slope test described in the section on construction. Working back from the measured energy grade lines, therefore, it was possible to compute all appropriate depths and velocities in the hypothetical model. These depths and velocities were then applied to the true inert elevations on the prototype to give the best values for hydraulic and energy grade lines and head losses. The three exceptions referred to above are the negative head losses between points 2 and 3 for minimum flows. Even the unadjusted data show something of this improbable situation, however, so it is not an indictment of the adjustment system, but rather of the accuracy of the original data. The amount of these discrepancies is on the order of 1/32 inch.

Corrections for H.G.L. at Point 2

At the peak dry flow it is desired that the North Central Outfall Sewer should not flow more than 3/4 full. Table II shows that the depth D_2 for combination I and VI at this flow for normal operating condition is 8.92 ft. With the invert elevation of 27.37 ft as it is in the model, this produces an energy grade line of 36.76 ft. In the prototype North Central Outfall Sewer, the invert elevation is actually 28.50 ft at point 2. Thus, to determine an effective depth for the prototype flow, we seek the depth which will produce a specific energy (depth plus velocity head) of 36.76 - 28.50 = 8.26 ft, with a discharge of Q = 357 cfs or discharge per unit width of q = 357/7.25 = 49.2 cfs per ft.

Table II. Depths, Velocities, and Head Losses
(Notes and Discharge Rates as in Table I)

Operating Condition	Flow Type	Point 1, u/s NOS				Point 3, Exist Prim. Channel				Head Loss 1 to 3
		Depth D_1 ft	Velocity V_1 fps	Spec. Energy E, ft	Energy Grade Line EGL_1	D_3 ft	V_3 fps	E_3 ft	EGL_3	
A	Max. storm	8.52	6.23	9.12	36.47	8.89	5.64	9.38	36.65	-0.18
	Pk. dry	8.74	5.05	9.14	36.49	8.88	5.38	9.33	36.60	-0.11
	Average	8.52	3.64	8.73	36.08	8.62	3.88	8.85	36.12	-0.04
	Minimum	7.32	0.82	7.33	34.68	7.45	0.80	7.46	34.73	-0.05
B	Pk. dry	8.30	5.31	8.74	36.09	8.67	4.13	8.94	36.21	-0.12
	Average	8.22	3.77	8.44	35.79	8.54	2.24	8.62	35.89	-0.10
C	Max. storm	8.62	6.16	9.21	36.56	8.90	5.96	9.45	36.72	-0.16
	Pk. dry	8.96	4.92	9.34	36.69	8.90	5.96	9.45	36.72	-0.03
	Average	9.20	3.37	9.38	36.73	8.89	5.64	9.38	36.65	0.08
	Minimum	7.56	0.79	7.57	34.92	7.63	1.49	7.67	34.94	-0.02
D	Average	8.68	3.58	8.86	36.23	9.11	1.57	9.15	36.42	0.19
	Minimum	6.84	0.87	6.85	34.20	6.95	0	6.95	34.22	-0.02

Table II - Continued

Operating Condition	Flow Type	Point 2, u/s NCOS				Head Loss 2 to 3	Point 4, New Prim. Channel				Head Loss 1 to 4
		D_2 ft	V_2 fps	E_2 ft	EGL_2		D_4 ft	V_4 fps	E_4 ft	EGL_4	
A	Max. storm	8.86	6.64	9.55	36.92	0.27	8.64	5.48	9.10	36.40	0.07
	Pk. dry	8.92	5.52	9.39	36.76	0.16	8.95	3.52	9.14	36.44	0.05
	Average	8.56	4.00	8.81	36.18	0.06	8.65	2.56	8.75	36.05	0.03
	Minimum	7.30	1.46	7.33	34.70	-0.03	7.35	1.07	7.37	34.67	0.01
B	Pk. dry	8.50	5.80	9.02	36.39	0.18	8.18	5.79	8.70	36.00	0.09
	Average	8.30	4.12	8.56	35.93	0.04	8.03	5.11	8.44	35.74	0.05
C	Max. storm	8.92	6.59	9.60	36.97	0.25	8.86	4.89	9.23	36.53	0.03
	Pk. dry	9.08	5.42	9.54	36.91	0.19	9.24	2.64	9.35	36.65	0.04
	Average	9.08	3.77	9.30	36.67	0.02	9.41	0	9.41	36.71	0.02
	Minimum	7.50	1.42	7.53	34.90	-0.04	9.61	0	9.61	34.91	0.01
D	Average	8.88	3.85	9.11	36.48	0.06	8.18	5.78	8.70	36.00	0.23
	Minimum	6.76	1.58	6.80	34.17	-0.05	6.85	2.30	6.93	34.23	-0.03

The relation to be solved is then:

$$D_2 + \frac{(49.2/D_2)^2}{2g} = 8.26 \text{ ft}$$

The solution gives $D_2 = 7.61$ ft or H.G.L. at (2) is 36.11 ft. As the soffit is to be 10.5 ft above the invert, the depth is seen to be 72.5 per cent of the height of the section, or under the maximum requirement of 3/4 full.

Development of Throat Design

Tail and Nose Combinations

Throat design includes the width of the throat, the tail piece which faces downstream, and the nose piece which faces upstream. Various combinations of tail and nose piece shapes were tested and are shown on Fig. 5. Hereafter they shall be referred to as T-1 for tail piece 1 and N-1 for nose piece 1.

The initial test of the model study was of the original interchange design submitted with Set I throat design. This design was found to pass all flows at normal operation satisfactorily to meet the requirements of the interchange. The point of interest then turned to the stagnation point which was observed to be pointing straight upstream (or a clock position of 12:00 on the nose piece) for peak dry and average flows of condition A. For condition B, average flow, the stagnation point moved to the throat side of the nose (2:00 position) and a deep depression or pocket of 0.4 ft prototype scale formed on the opposite side of the nose followed by considerable turbulence. The depth, size, and turbulence of the pocket became more pronounced for condition D, average flow.

The first effort to improve the original throat design was the insertion of a longer tail, T-2, curved away from the throat, restricting the NCOS mouth opening to 80 per cent of the full channel width. T-2 produced two major effects. The hydraulic grade line at point 1 was lowered and that at point 2 in the North Central Outfall Sewer was raised. Thus, a greater amount of energy was transferred by turbulent shear from the high velocity flow coming from the North Central Outfall Sewer to the low velocity flow coming through the throat, enabling a lower water depth at the throat and consequently at point 1. Set II also produced a more markedly disturbed water surface downstream of the interchange, where the turbulent exchange of energy took place. Before discarding Set II because of this turbulence, it was tried with a long radius nose piece, N-2, which produced no significant difference from the Set II combination.

The next step at modification was with a short tail piece, T-3, and long radius nose piece, N-2. This combination provided the widest throat opening, but showed no interchange requirements improvement for normal operation. However, for condition C, average flow, where the maximum of flow passes through the throat, Set IV produced a lower HGL at point 1 than Set I throat design under similar operating conditions. The major factor in its disapproval was the greater tendency for the flow from the North Central Outfall Sewer to enter the new primary influent channel.

Final Throat Design, Set V

In the long radius nose piece, N-2, there was one feature that was desirable. Under abnormal operating conditions, with unnatural distribution of

flows between the throat and the new primary influent channel, the stagnation point remained on the rounded nose even though it shifted in orientation. The flow was therefore smoother and a much shallower pocket was formed. The nose piece finally chosen with a shorter (1 ft) radius preserves this feature. The T-1, N-3 throat design therefore, does not represent an improvement over the T-1, N-1 combination for flows at normal operations, but it is an improvement for almost all of the abnormal operations.

Figure 6 shows a dye test at normal operation, peak dry flow. A slug of dye was injected into the North Central Outfall Sewer and it may be seen that none of the dye is entering the new primary influent channel. In fact it took a distance of a foot or more in the model before the dye completely mixed with the flow in the existing influent channel.

Figure 7 illustrates the use of tracer particles of gilsonite (an asphaltic mineral having specific gravity of $1.04\pm$) with a 1/25-second exposure. At normal operation, average flow, the particles are seen to separate smoothly between the throat and the new primary influent channel. A slight eddy and depression of the water surface was observed in the lee of the nose, but the surface was not broken by turbulence at normal operation, maximum storm flow.

Figure 8 illustrates condition B, average flow, for which there is no net flow through the throat. The 1/25-second exposure of Fig. 8 shows that the particles are not passing through the throat. A smooth depression of the surface in the lee of the nose was observed during the test. Closer observation of the throat at this flow revealed that there is actually a small downstream flow at the surface and a corresponding back flow along the invert. This motion was sufficiently strong in the model to prevent any deposition of sediment.

Figure 9 shows abnormal condition C, average flow, for which the flow through the throat is a maximum. The particle cloud of Fig. 9 reveals the manner in which the flow leaves the outer wall just before the throat, with a clockwise eddy just at the point of departure. A slight depression formed on the throat side of the nose as the flow was accelerated at this point. There is some waviness of the surface downstream of the throat which is inevitable.

Analysis of Special Features

Mechanism of Energy Transfer from Q_2 to Q_5

It is instructive to investigate the mechanism by which energy is transferred from the North Central Outfall Sewer (Q_2) to the slower moving flow through the throat (Q_5), so that the energy grade line at point 3 is 0.11 ft higher than that at point 1 for condition A, peak dry flow (with throat design V). Table II shows that the head loss from (2) to (3) is 0.16 ft. As Q_2 is 357 cfs (230 mgd), this represents a power loss from (2) to (3) of

$$0.16 \times 357 \times 62.4 = 3{,}560 \text{ ft lb/sec.}$$

For Q_5, on the other hand, there is a gain of 0.11 ft from (1) to (3), and, for Q_5 of 263 cfs, this represents a power gain of

$$0.11 \times 263 \times 62.4 = 1{,}803 \text{ ft lb/sec.}$$

By subtraction, the net power loss in the two components of Q_3 between

Fig. 6 - Dye Test, Throat Design V,
Condition A, Peak Dry Flow

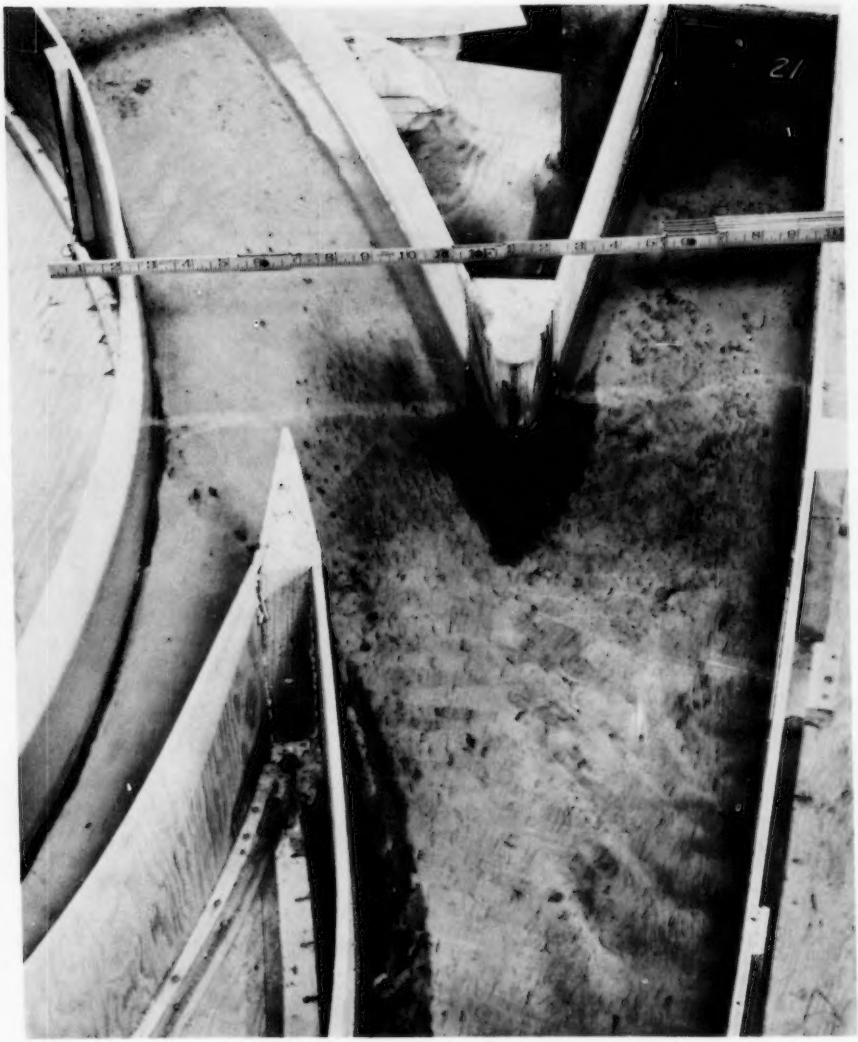

Fig. 7 - Throat Design V, Condition A,
Average Flow

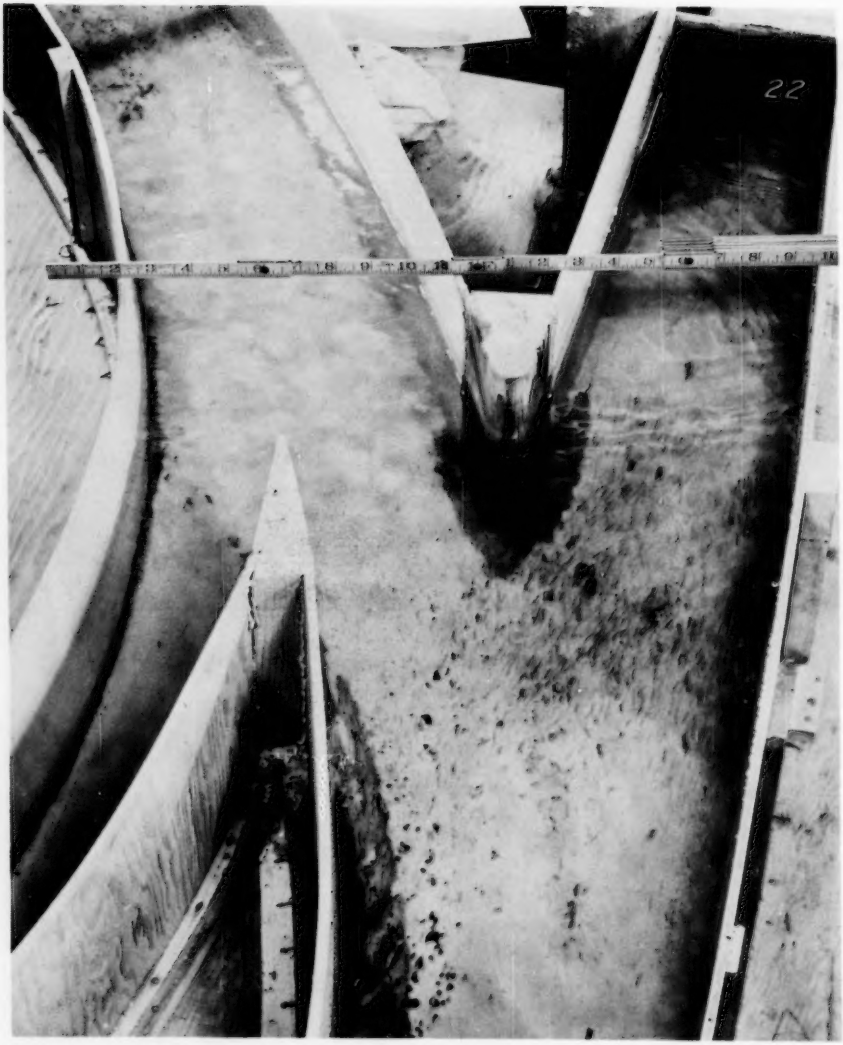

Fig. 8 - Throat Design V, Condition B,
Average Flow

Fig. 9 - Throat Design V, Abnormal Condition C, Average Flow

points 1 or 2 is 1,757 ft lb/sec. For Q_3 of 620 cfs, this means a head loss of 0.045 ft.

This transfer of energy must be accomplished by a velocity gradient and corresponding shear between Q_2 and Q_5. At the mouth of the North Central Outfall Sewer the corresponding mean velocities are computed to be,

$$V_2 = 5.52 \text{ ft/sec}$$

$$V_5 = \underline{2.87}$$

$$\Delta V = 2.65 \text{ ft/sec}$$

If the zone of shear may be roughly estimated to be 2 ft wide, the velocity gradient would be

$$du/dy = 2.65/2 = 1.325 \text{ sec}^{-1}$$

To investigate the magnitude of the turbulent shear, consider the Prandtl equation which includes mixing length, ℓ

$$\tau = \mu \frac{du}{dy} + \rho \ell^2 \left(\frac{du}{dy}\right)^2 \qquad (7)$$

The first term represents the viscous shear and is seen to be a negligible quantity. Evaluating the second term, assuming that the mixing length is also approximately 2 ft, and using $\rho = 1.94$ slug/cu ft for the density of water,

$$\tau = 1.94 \times 2^2 \times (1.325)^2 = 13.6 \text{ lb/ft}^2$$

If we assume that the length of this shearing region is on the order of 10 ft, we have a contact area of this length times the depth, or approximately 10 x 8.5 or 85 ft^2. The total shearing force would then be on the order of 85 x 13.6 or 1,156 lb. If this force is applied to Q_5, moving at about 2.9 ft/sec, the rate of energy transfer would be about

$$1{,}156 \times 2.9 \text{ or } 3{,}350 \text{ ft lb/sec.}$$

Now, if it be assumed that the loss in energy of Q_2 from (2) to (3) owing to wall friction and turbulence is (from above calculations) about

$$.045 \times 357 \times 62.4 = 1{,}000 \text{ ft lb/sec,}$$

this would mean that the rate of energy transfer from Q_2 to Q_5 would be on the order of 3,560 - 1,000 or 2,560 ft lb/sec.

This is seen to be about three fourths of what the tentative shear calculation above showed. The assumed values for velocity gradient or mixing length or shear area are evidently too high.

The phenomenon of negative head loss from a lateral to a conduit in combining flow has been observed in closed conduit flow by Prof. J. S. McNown, M. ASCE[4], but in this case the lateral flow entered at right angles to the main conduit and there could be little or no opportunity for the type of energy transfer discussed above. The negative head loss was, therefore, restricted to

4. "Mechanics of Manifold Flow," by John S. McNown, Trans. ASCE, Vol. 119, 1954, p. 1117.

lateral flows of less than 15 per cent of the combined flow, and was attributed to the lateral flow moving into a region of the conduit where the velocity was below average.

Deposit of Sediment

During the model tests some fine sand was injected into the North Outfall Sewer for visual observation. This was found to settle out on the bottom at the lower flow rates, and it may be seen along the convex curving wall of the North Outfall Sewer upstream from the interchange in Figs. 7, 8, and 9. At practically all flows there is some separation and back flow along this wall, as the flow decelerates in the widening channel.

In an effort to minimize this deposition, a bulge was placed in the right wall of the North Outfall Sewer just opposite the interchange extending 2.5 ft into the stream, and smoothly faired so that the North Outfall Sewer in effect did not diverge until almost reaching the interchange proper. The sediment was scoured away, but the bulge produced a highly disturbed flow in the new primary influent channel. This was not considered to be a practicable solution to the deposition problem. In fact it should be emphasized that prototype velocities will be 4.47 times those of the model, and sand such as was used in the model will certainly be scoured. In the region of the throat, even if there is not much flow through the throat there is generally enough circulatory motion to keep light material on the bottom in suspension. If deposition does occur at the low flows, moreover, it may easily be scoured by operating temporarily so as to sluice one channel or another with higher velocity flow.

Extension to Plant Operation

In order to attain an efficient operating condition at the existing plant, artificial devices for head losses must be utilized for all but maximum flows. These devices for backing up incoming flows consist of sluice gates and stop log weirs and they are used separately or together depending on the amount of the flow. Specification No. 2 of the Introduction sets the permissible depth of backwater allowed with or without the interchange at peak dry and maximum flow, normal condition A.

In the proposed operation of the Hyperion Sewage Treatment Plant, it is desired to regulate the flows to the existing and new primaries by the throttling of a butterfly valve located immediately downstream from a venturi tube in the new primary influent channel. The intelligence to position the valve will be furnished by a ratio controller located in the control house. Information required for proper division of flows by the controller will be furnished by a three unit summator which receives signals from a transmitter at the new venturi tube and from two transmitters at the two existing venturi tubes.

The experimental data furnished by the model showed that the proposed operation of the plant may be reasonably accomplished for normal operation. However, for abnormal conditions C and D, average flows, the existing sluice gates and stop log weirs would have to be throttled in order to produce a sufficiently high HGL at point 4 to pass the flows through the new primary. Of course it is conceivable to set the new primary tanks sufficiently low that even these flows can be passed without throttling the existing sluice gates and stop log weirs. This would, however, indicate a low weir elevation that would be undesirable from the viewpoint of the overall plant enlargement.

CONCLUSIONS

Of the five designs of throat tested in the 1:20 scale model of the Hyperion sewer interchange, the combination of a long slender tail and a 1-ft radius nose (Set V of Fig. 5) approached most closely the criteria set forth in the Introduction. The results of the model study with this throat design, projected to prototype dimensions and conditions, yield the following conclusions:

1. For all conditions where there is a positive flow through the throat, sewage in the North Central Outfall Sewer is excluded from the new primary influent channel. This was demonstrated by injecting dye in the North Central Outfall Sewer and observing that no color appeared in the new primary influent channel (see Fig. 6). Thus, a maximum of the saline sewage from the North Outfall Sewer will flow to the new primary.

2. For normal operation the North Outfall Sewer is not backed up to a hydraulic grade line at point 1 greater than 36.1 ft. This value occurs at peak dry flow. Maximum storm and average flows each produce an HGL of 35.9 ft at point 1. These figures include an allowance of about 0.4 ft for head loss across bar screens or other obstructions at the inlet to the existing detritors.

3. The water surface in the interchange is relatively smooth. Under normal operating condition this is also accomplished with Set I throat design, but for the abnormal conditions the cylindrical nose on the diversion pier is especially helpful.

4. If deposition of sediment occurs in the interchange at all, it will most likely occur along the convex or northeast wall of the North Outfall Sewer, just upstream from the confluence with the North Central Outfall Sewer where there is some separation. Accompanying the separation, however, there is a circulatory flow, probably enough to keep most sediment scoured from the bottom.

5. Various combinations of throat design other than the recommended one each displayed certain attributes but in every case the accompanying disadvantages outweighed the advantages. The original or Set I design was found to be satisfactory for normal operation, but for most of the abnormal conditions the cylindrical nose on the diversion pier produced a smoother water surface and a lower head loss across the throat.

ACKNOWLEDGMENTS

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FLOW OF CONCENTRATED RAW SEWAGE SLUDGES IN PIPES

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(Proc. Paper 1274)

INTRODUCTION

Concentrating raw sewage sludges to 8 or 12% total solids on a continuous basis has introduced to the sewage treatment field an attendant problem of handling the concentrate. A recent study reported herein indicates that with proper design consideration concentrated raw sludges can be pumped without difficulty.

The purpose of the study was to obtain information for the design of sewage sludge pipe lines. Two particular objectives were to determine the relationship of flow resistance and solids concentration and to determine if any empirical connection existed between the commonly used hydraulic formula established by Hazen and Williams and the flow characteristics of concentrated sludges. The basic factors that affect flow behavior such as viscosity, temperature and velocity were observed generally.

Experience has shown that primary sewage sludges can be concentrated to a greater extent than secondary sewage sludges or mixtures of primary and secondary sludges. To take advantage of high solids concentrations for testing purposes, sludge flow information was collected on highly concentrated sludges produced during a Densludge pilot plant study at the Grass Island Primary Treatment Plant in Greenwich, Connecticut.

Previous Work

A search of previous work reveals that considerable variance and difficulty have been encountered in observing the performance of sewage sludges under flowing conditions. Referring to the flow of sewage sludges in pipes, Eddy stated in 1924:

> "The first series of experiments that had been tried in the United States were under Mr. Holmes's direction . . . . . Unfortunately, the results were practically negative . . . . . due to almost insurmountable difficulties in making the tests."(1)

Note: Discussion open until November 1, 1957. Paper 1274 is part of the copyrighted Journal of the Sanitary Engineering Division of the American Society of Civil Engineers, Vol. 83, No. SA 3, June, 1957.

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In 1929, a committee of the American Society of Civil Engineers reported the results of extensive research and study of the flow of sewage sludge.(2) The conclusions reached then are repeated below because they have continued to provide a basis for present thinking.

"1. Sludge is neither a viscous nor a homogeneous material, but is variable in character.

2. The usual analytical tests do not define its physical qualities, but it seems to behave more like suspended matter.

3. Below the critical velocity, sludge has a different friction factor from that above. As yet the coefficient of flow below the critical velocity, cannot be concisely stated. Above the critical velocity, it can only be expressed in ranges.

4. Sludge friction losses increase with a decrease of moisture content.

5. Sludge friction losses tend to increase with lower temperatures.

6. Sludge friction losses for high velocities (from about 5 to 6 ft. per sec., or more), tend to follow more nearly the characteristic laws for the flow of water.

7. Friction losses for fresh or undigested sludge and for sludge from combined sewage are more erratic and the determination of a friction factor is correspondingly more difficult to obtain.

8. Within the limits investigated, no law of flow has been found."

Further work by Babbitt and Caldwell extensively covered laminar and turbulent flow of sludges in pipes as referred to in conclusion 3 above.(3, 4) Babbitt and Caldwell concluded that there are two "critical" velocities: a lower velocity below which laminar flow and formulae apply, and a higher velocity above which turbulent flow and exponential formulae such as Hazen and Williams' apply; and between the lower and upper critical velocities, either laminar or turbulent flow may exist. It was stipulated, however, that for the turbulent formulae to apply, "the sludge must be made up of finely divided substances disbursed in water and the velocity of flow must be above critical."

Hatfield studied the "Viscosity or Pseudo-plastic Properties of Sewage Sludges," and provided extensive basic study. During his work, he encountered the thixotropic properties of sludges which prevent accurate viscosity predictions. A thixotropic material may be defined as a mixture which has a tendency to jell, liquify on stirring, and jell again on standing.

In some of the works referred to above, little differentiation was made between raw or digested sewage sludges. Also, some work was done with clay or other sludges, and the results of such tests were extended for use of all sludges, including sewage.

Greenwich Tests

In comparative observations during the tests at Greenwich, Connecticut, raw sludge exhibited considerably more resistance to flow than did digested sludge having the same solids concentration. Observations also indicated that once raw solids are concentrated in a thickener tank, there is little, if any,

segregation of the concentrated product after further standing. Thus, there need be little concern over reduction of pipeline capacity as a result of solids settling in the pipeline at low or below critical (non-scouring) velocity. The existence of such a situation is indeed fortunate because it is not necessary to maintain turbulent (scouring) velocities while pumping thickened raw sludges, and thus to incur unnecessarily high head losses. Although grease accumulations may possibly deplete pipe capacity for either thin or concentrated sludges, no serious grease coatings were found on the pipe walls at Greenwich after three months' operation.

Installation

The test installation included a source of concentrated raw sludge, a source of digested sludge, two sludge pumps, and a pipeline from these sources to the pump and beyond to discharge at the plant digester. The two sludge pumps, an Oliver Diaphragm Slurry Pump and a Type SA Dorrco Plunger Pump, were installed in parallel to pump through a common pipe to the plant digester. These pumps were usually used on an alternate basis.

Pressure gauges as shown in Figure A were installed on each side of various fittings and at the ends of straight lengths of pipe 10 and 20 feet long. Originally there was some anticipation that the small opening in the pressure gauges would become clogged by the thick sludge. Use of the small chamber shown in Figure A proved satisfactory in preventing clogging and transmitting pressure to the gauges. Two cocks were provided: one between the chamber and gauge; the other to allow flushing or the addition of compressed air to the chambers, whichever was required. No trouble was experienced with clogged gauges throughout the three months of tests.

Consideration was given to the use of air or clean liquid as a buffer in the chamber between the sludge and the gauge entrance. The compressibility of air was noted and its effects contemplated. After brief trial entrapped air was employed, primarily because it facilitated gauge readings during the pulsating flow produced by the positive displacement pump. Further, because differences were dealt with and all gauges operated under the same circumstances, any inaccuracies resulting from gas compression appeared to be small. Before and after each test run, readings were made of all test gauges under static conditions and proved to be consistently equal to each other.

The majority of the piping was 4" welded steel. One 20' test section was flanged to allow replacement by different sized pipe. A 6" section was later installed and tested.

Test Procedure

In general, the procedure was to take simultaneous pressure gauge readings at the different points along the pipeline while sludge was being pumped. Samples of the sludge were taken and composited for each group of tests. The composite samples were then analyzed for total and volatile solids. On several occasions the individual grab samples were also analyzed for total solids and seldom revealed more than 3% variance from the average.

The volume of pumpage was determined by two different methods which were checked against each other. One method was to calculate the volume by measuring the number of strokes and multiplying these by the capacity per

stroke. The second method was to measure the drawdown in the sludge storage tank, calculate the volume, and relate this to the time interval or number of strokes pumped.

The flow from either the diaphragm or plunger pump was not continuous but of a pulsating nature, occurring even though there were air chambers on the discharge side of the pump. The pulsating flow caused the pressure gauge readings to flutter from high to low in a cyclic pattern. These cyclic fluctuations are illustrated in Figure B.

The air chambers and friction along the pipeline progressively dampened the pulsations, and the degree of fluctuation became less pronounced with each successive gauge. Thus the pulsating effect lessened as the length of pipe increased but was always evident throughout the entire length of the 60' section tested.

The problem of fluctuating gauge readings produced by the pulsating flow caused some skepticism that useful data could be obtained. However, as positive displacement pumps are generally employed to handle thick sludges, and because of the sparseness of published data on this subject, it was obvious that typical operating data of any type would be of value.

In order to follow the surging pattern and obtain instantaneous readings at each gauge simultaneously, the gauges were read at the maximum pressure for each stroke. The nature of the cycle was such that the taking of this reading was easy. A number of such readings were made for each test run, and differences between gauges were determined and then averaged. The head losses per length of pipe decreased as the distance from the pump increased. In other words, the head loss adjacent to a reciprocating pump is a function of the instantaneous surge rate, and the head loss at a great distance from the pump is a function (more or less) of the average rate of pumpage. This observation suggests the possibility of installing pipelines with small air chambers and decreasing diameters.

Because it is undesirable and unlikely in practice to have a sludge line of the length required to obtain non-pulsating flow, the average head losses obtained over the major twenty-foot test section are considered suitable for design purposes and are presented in Figures C & D. Figure C presents data for the 4" steel pipe and shows the relative effect of velocity. Head losses for intermediate velocities should fall between these lines. The surge flow rates at which measurements were made have been estimated at 200 gallons per minute for the ODS pump at 6 gallons per stroke, and 60 gallons per minute for the Dorrco Plunger Pump when producing 0.4 gallons per stroke.

The intercept at 0% total solids for the 5.08 fps line checks closely with the theoretical when using a surge rate of 200 gallons per minute and a Hazen-Williams Coefficient of 100.

Figure D presents data for 6" steel pipe. Actual measurements were obtained with more concentrated sludges only. Below 6% solids, measurable differences could not be obtained. Consequently, a theoretical calculation was made for 0% solids, and this point was used in joining the other data.

When using either Figure C or D for design of pulsating pump installations, the surge velocity must be used. Use of average velocities for a pulsating flow may result in pipe lines of too small capacity.

Discussion

According to Babbitt and Caldwell,[3] in order that formulae such as Hazen and Williams' may be applied, "the sludge must be made up of finely divided substance dispersed in water, and the velocity of flow must be above critical." The sludges used in this study seldom met this definition in a strict sense. However, because the Hazen-Williams formulae and nomograph are so widely used, an effort was made to determine if there were any empirical correlation between the increase of solids concentration and an "apparent" decrease in the coefficient, C. A coefficient of 100 was used as a basis after it was determined by the plotted intercept at 0% solids. Using the same flow quantity and the observed head losses, the apparent "C" value (i.e. a value that may be substituted for C) was calculated for different concentrations above 0% solids. The results are presented below in Table A.

TABLE A

EFFECT OF SOLIDS CONCENTRATION ON HYDRAULIC COEFFICIENT

Percent Total Solids Raw Sludge	Apparent Hazen Williams Coefficient C					Percent less than Basis
	4-inch Pipe		6-inch Pipe			
	5.08 fps	1.53fps	2.25 fps	0.67 fps	Av.	
0	100*	100*	100*	100*	100*	-
2	85	80	80	80	81	19
4	60	55	60	70	61	39
6	50	35	40	55	45	55
8.5	37	28	30	32	32	68
10	32	20	25	25	25	75

*basis

Greely and Stanley state: "Sewage sludge does not constitute a homogenous fluid and the more common hydraulic laws can be applied only approximately. . . . Present data do not permit close computations of the hydraulics in handling sludge and empirical methods based upon experience are generally used. . . . The pipe friction losses of sludge may range from 1.5 to 4 times the losses for water."(6)

Babbitt states: "If velocities above critical are used, the friction loss in the sludge pipe can be estimated by the application of the Hazen-Williams formulae using values of C 20-40% less than values that have been established for the flow of water in pipes of the material to be used for conveying sludge."(7)

The above references describe generally the practice of design followed to date. The Greenwich tests confirm that such approximations were adequate for raw sludges of concentration up to the range between 4 and 6% solids, this being the then general limit of concentrations pumped. This study extends the knowledge for design applications when concentrated raw sludges in the order of 8 to 10% total solids are handled and indicates that the pipe friction loss in

this higher range of solids concentration may be 6 to 8 times the loss of water. Friction losses can be estimated with the Hazen-Williams formula using C values 60 to 75% less than those established with the flow of water in a pipe of the same material.

Head loss data was also obtained for a standard 4" weld elbow. By comparing the data on head loss in the elbow with a concurrent head loss in the 4" pipe, an equivalent length of pipe was obtained for several test runs. The results are presented below in Table B.

TABLE B

HEAD LOSS DATA 4-INCH ELBOW

	1.53 fps		5.08 fps
% T.S.	Equiv. foot 4-inch pipe	% T.S.	Equiv. foot 4-inch pipe
6.0	18.8	0.9	13.5
6.2	11.7	2.6	13.8
6.5	4.9	4.8	11.3
7.15	5.4	6.0	9.6
7.15	11.8	8.15	6.1
7.2	9.2	8.2	4.5
7.75	9.4	9.0	5.0
7.75	6.4	9.1	4.3
8.0	3.3		
8.15	7.8	Average	8.5
8.5	3.5		
8.6	6.7		
8.6	2.7		
Average	7.8		

The above data, as well as the plotted data in Figures C and D, show that variations were encountered but may be expected when one considers the nature of the material being pumped.

The average equivalent length of 4-inch pipe required to give the same head loss as a 4-inch elbow is about 8 feet. Reference to a commonly used nomograph of "Resistance of Valves and Fittings to Flow of Fluids," shows the equivalent length of straight pipe for a medium sweep elbow to be 8.5 feet.(8) This singular agreement indicates that one may, for design purposes, use the same equivalent lengths of pipe as for those cases when pure liquid is used. Once the equivalent length is determined, the proper head loss per foot may be applied, depending upon the concentration of the sludge to be pumped.

The solids concentration of raw sludge was the only variable studied in detail during the tests. Other influential factors noted by previous investigators were generally observed. The heterogeneous nature was certainly evident. The thixotropic property was also noted. A gentle continuous agitation of the sludge is provided in the sludge thickener, and a sample drawn after pumpage and left standing for a period of time will, on occasion, exhibit a jell.

Temperature appeared to affect the fluidity of the sludge more than any other variable except solids concentration. The average temperature of the sludge during the test reported was 65°F. A decrease in temperature below 65°F appeared to increase the head losses considerably. In contrast, a slight degree of increase above 65°F in temperature appeared to have less influence in decreasing the head losses. Little variance of volatile content from 78% was encountered, and consequently no effect was observed.

Although head loss data when pumping digested sludge was obtained in only a few cases, the digested sludge appeared to be much more fluid in characteristics than raw sludge. A digested sludge gave less head loss per foot of pipe than a raw sludge of the same solids concentration.

Pumps

Both of the pumps used proved to be entirely adequate for pumping sewage sludge of the concentrations tested which were up to 16% total solids. The diaphragm (O.D.S.) pump was able to obtain a higher suction, however, being limited only by the capacity of the vacuum pump applied.

SUMMARY

The flow of concentrated raw sewage through pipes was studied in conjunction with a Densludge pilot plant at Greenwich, Connecticut. The results coincide with and extend the work of previous investigators. When concentrated raw sludges in the order of 8 to 10% total solids are handled, the pipe friction losses may be six to eight times the losses when water is used. In the same manner, friction losses can be estimated with the Hazen-Williams formula using C values 60 to 75% less than those established with the flow of water in a pipe of the same material.

No difficulty was experienced in pumping solids up to a concentration of 16% total solids with either of the two pumps used.

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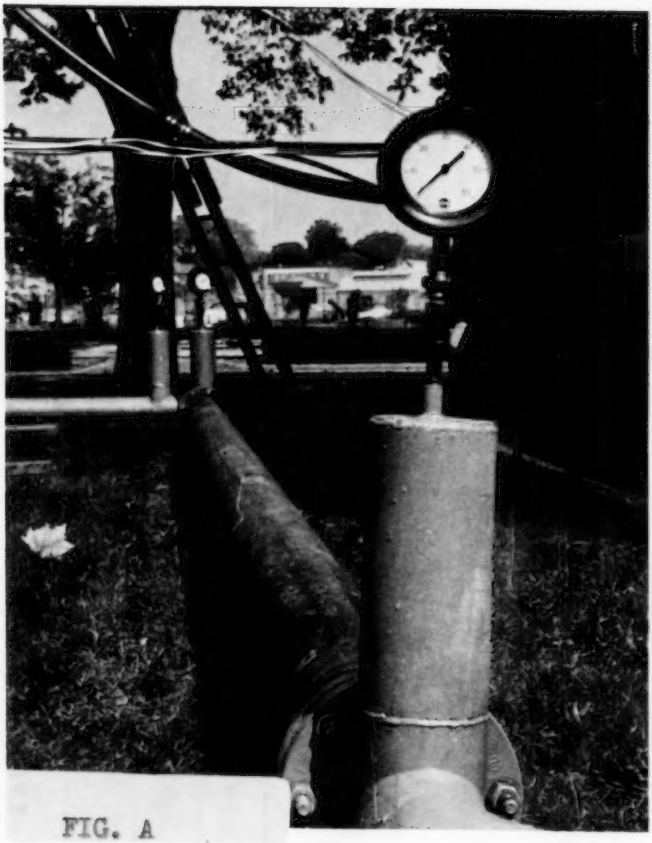

FIG. A

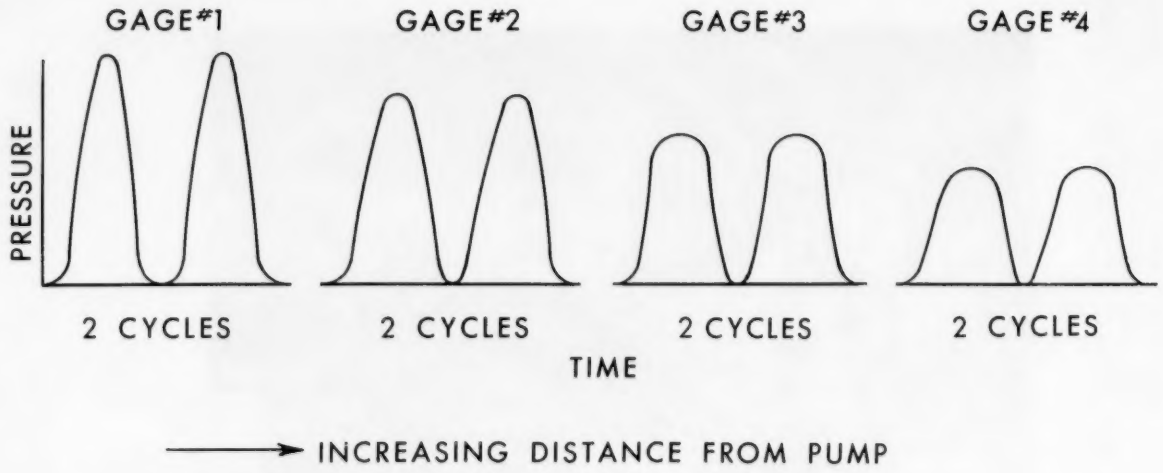

FIG. B
TYPICAL GAGE FLUCTUATIONS ALONG TEST LENGTH

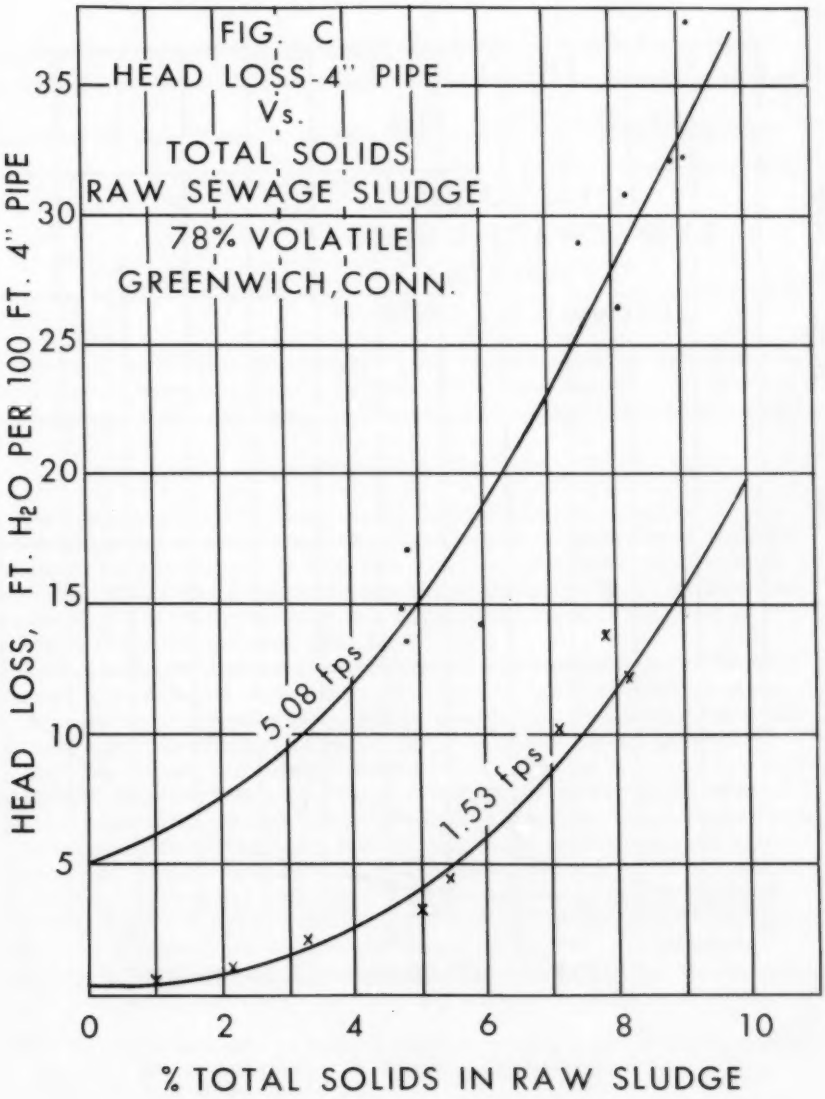
FIG. C
HEAD LOSS-4" PIPE
Vs.
TOTAL SOLIDS
RAW SEWAGE SLUDGE
78% VOLATILE
GREENWICH, CONN.
HEAD LOSS, FT. H_2O PER 100 FT. 4" PIPE
35
30
25
20
15
10
5
5.08 fps
1.53 fps
0
2
4
6
8
10
% TOTAL SOLIDS IN RAW SLUDGE

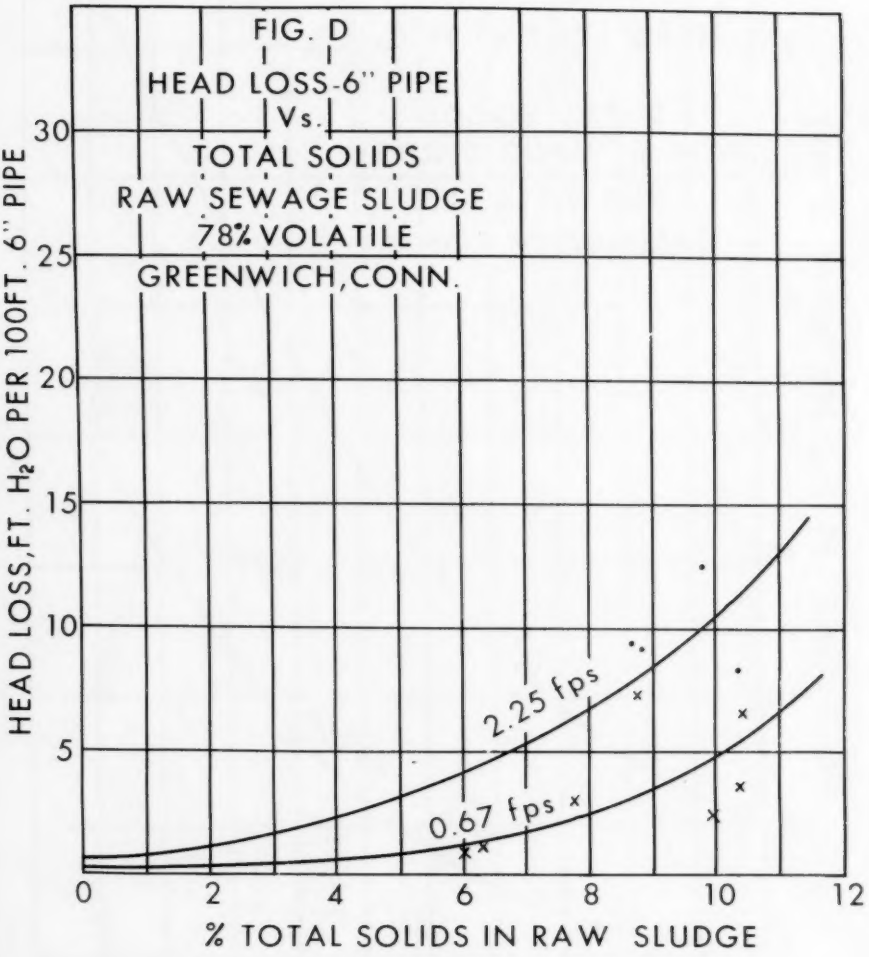
FIG. D
HEAD LOSS-6" PIPE
Vs.
TOTAL SOLIDS
RAW SEWAGE SLUDGE
78% VOLATILE
GREENWICH, CONN.
HEAD LOSS, FT. H_2O PER 100FT. 6" PIPE
30
25
20
15
10
5
0
2
4
6
8
10
12
% TOTAL SOLIDS IN RAW SLUDGE
2.25 fps
0.67 fps

Paper 1275 SA 3

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RECENT DEVELOPMENTS IN THE TREATMENT OF ATOMIC WASTES*

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(Proc. Paper 1275)

SYNOPSIS

The handling of the process wastes from the atomic energy industry pose some unusual waste treatment problems. This paper discusses the developments for handling, treatment, and disposal of the radioactive liquid wastes.

The waste materials from the atomic energy industry are gaseous, liquid, and solid and occur in any phase of the industry from the mining of the uranium ore to the ultimate use of a specific radioisotope in industry, research, or medicine. These waste materials differ from those with which we have been concerned in the past in that they are radioactive. As such they may be damaging to human and other tissues.

In this paper the discussion will be limited to a consideration of the developments and proposals made for the handling, treatment or disposal of the radioactive liquid wastes. The problems associated with the handling of the gaseous and solid wastes have been discussed by Wolman and Gorman.(1)

For convenience, although the breakdown is somewhat arbitrary, the problems of liquid waste management will be considered under three headings: Low, intermediate, and high level wastes. Low level wastes are those which, if decontaminated by a factor of 100 or 1000, would approach permissible limits(2) for human exposure. The range of their activity would be from 10^{-4} to 10^{-3} microcuries per milliliter. High level wastes are those requiring shielding to protect persons handling them from exposure to damaging radiation and may contain 100 or more curies per liter. They are generally associated with the chemical processing of the nuclear fuel for the recovery of the fissionable materials. Intermediate wastes also require shielding and

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must be handled with considerable care. They may be the high level wastes which have lost a considerable fraction of their activity as a result of decay, the residual wastes following recovery or separation of the strontium or cesium isotopes, the wastes resulting from pilot plant operation, or low level wastes that may have been concentrated by evaporation or other means.

Low Level Wastes

Low level wastes are those most frequently encountered at the present time at least insofar as release into the environment is concerned. Depending upon their source, they are handled or disposed of in various ways. They may be contained in the residues following the recovery of uranium ore in which case they will contain essentially the daughter products of the uranium. Generally, these waste materials are discharged into the stream either directly or from tailings ponds. Studies by the Public Health Service have shown that water, mud, and biological samples collected below uranium mills have activity levels higher than similar samples collected upstream.(15) The public health significance of these higher levels has not been evaluated as yet.

Another source of low level wastes results from the use of natural waters as reactor coolants. In passing through the reactor, the chemical constituents of the water from natural sources or corrosion products undergo neutron bombardment, and become radioactive. Most of this activity is relatively short-lived and is associated with the lighter elements. The largest source of such wastes is the Atomic Energy Installation at Hanford where pre-treated Columbia River water is used to cool the reactors. Before return to the Columbia River, the water passes through specially designed retention tanks which permit maximum decay of the radioactivity.

Low level wastes are produced in any laboratory where radioactive materials are used. They are extremely variable in composition—both from the standpoint of their chemical as well as radioactive constituents—and are generally released into the environment following limited treatment. At Oak Ridge National Laboratory, for example, they pass through settling basins before release into the Tennessee Valley System through the Clinch River. In general, the activity level in the Clinch River below the plant discharge is at the 10^{-7} microcurie per milliliter level indicated for mixed fission products or activities of unknown composition. Plans are underway to construct a treatment plant for the removal of strontium and other activities present in the wastes during peak activity levels. The plant is flexible in design and will permit the use of either phosphate coagulation or lime-soda ash softening to precipitate the strontium before the wastes are released to the Clinch River.

Brookhaven National Laboratory concentrates its waste by evaporation, mixes it with concrete, places it in steel drums, and carries it out to sea for disposal.

In Great Britain, the low level wastes which arise from the Atomic Energy Research Establishment at Harwell are discharged into the Thames River following treatment consisting of chemical coagulation. Treatment must be such that the quantities of radioactive materials discharged not exceed 20 curies per month or 5 curies in any one day as determined by the formula:

Ra (curies x 2500 + Other Alpha (curies) x 420 + (Ca++ + Sr++) (curies) x 50 + Remaining Beta (curies) $\leq$ 20 (curies) per month.

This formula was arrived at by applying a factor of 1/100 to the maximum permissible concentrations recommended by the International Commission on Radiological Protection.(3)*

Burns(4) reports that a small plant has been in operation at Harwell for some time treating mixed fission product solution in water of a low solids content. By treating this waste with phosphate and iron salts followed by a sulfide treatment and passing the liquid through columns of Vermiculite, decontamination factors of the order of 1000 have been obtained. If necessary, the water could be treated further by an electric deionization process which is expected to increase the decontamination factor to 100,000. If this is achieved it should be possible to reuse the water for many purposes and thus effect an economy.

Other formulas were developed for use in connection with the discharge of radioactive materials from the operations at Windscale, near Sellafield, England. Here the limiting levels for discharge into the Irish Sea were determined by the accumulation of activity in fish, in an edible sea weed, and on the beach sands. The formulas are:

$$\frac{\text{Total Alpha (curies)}}{200} + \frac{\text{Ru (curies)}}{5000} + \frac{\text{Total Beta (curies)}}{20000} \leqslant 1 \text{ (curie) per month}$$

$$\frac{\text{Sr (curies)}}{2500} + \frac{\text{Total Alpha (curies)}}{300} \leqslant 1 \text{ (curie) per month.}$$

The most common method of disposal, particularly by users of radioactive isotopes following application in medicine, research, or industry, is to discharge these materials into the sewer. Although this practice does not appear to be objectionable at present, it may become necessary for public health authorities to maintain vigilance over the quantities of radioactive materials discharged as the use of radioisotopes increases.

Considerable information has been published on the effectiveness of water treatment methods for the removal of low level radioactive contamination. It will not be reviewed here, but the principles involved may be applied successfully to the treatment of low level wastes. Methods that may be used include coagulation,(16,17) softening,(16,17) ion exchange,(16,17,18) permselective membranes,(19,20) clays,(17,21) powdered metals,(22) etc.

Intermediate Level Wastes

In the past, storage was used for the retention of all liquid radioactive wastes containing appreciable quantities of radioactive materials. More recently, consideration has been given to the release of radioactive materials into the ground and into the oceans. If this can be accomplished without hazard to the environment, the cost of waste treatment and disposal may be materially reduced.

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* The British selected a factor of 1/100 because of the large numbers of people located downstream on the Thames River below the Harwell Establishment. This factor may be compared with a value of 1/10 used in this country when recommending levels for the general population.

Ground Disposal

This method has been practiced at the Hanford site for approximately 10 years. Under the rather unique geologic, hydrologic and meteorologic conditions at this site approximately 109 liters of active waste containing several hundred thousand curies of fission products have been released into the ground for storage.(5) At the Oak Ridge National Laboratory, four waste pits have been scooped out in the weathered Conasauga shale overburden and have received the quantities of waste reported in Table I.(6) Experience indicates that the nitrate moves through the soil most rapidly followed by the anionic ruthenium present.

In the use of the ground, as will be defined more specifically under high level wastes, one must always be certain that the amount of radioactive material transmitted to the ground water will be at a sufficiently low level that the maximum permissible concentration levels will not be exceeded in its use as a source of water supply. Furthermore, it must be remembered that once radioactive materials are introduced into the ground, control of them has been lost.

Ocean Disposal

Except for the reference made to the Windscale operations in England little if any radioactive liquid wastes are discharged directly into the oceans. Some materials are discharged but these are in packaged form. The radioactive materials, in the form of evaporator concentrates, precipitated and dewatered sludges, etc., are mixed with concrete directly or placed in previously prepared concrete shielded cylinders, and then shipped out to sea for disposal. It has been reported(1) that approximately 20 to 30 tons of waste are disposed of annually on the East Coast as compared to 4 to 5 tons on the West Coast. Disposal must take place beyond the 1000 fathom level(7) in this country, and the British disposals are carried out in water more than 2000 fathoms deep, several hundred miles from the British Coast, and in areas approved by the Minister of Agriculture, Fisheries and Food.(8) The need for special reinforcing of the concrete in the containers to provide strength to meet the impact when the container strikes the hard ocean floor and the high hydrostatic pressures in 500 or 1000 fathoms of water was pointed out.(1)

High Level Wastes

These originate principally from the chemical processing of spent fuel for the recovery of the fissionable material. Associated with the irradiated fuel is the entire spectrum of fission products which occurs when the uranium atoms are split. The fission products must be separated from the fuel to permit further processing or reuse of the fuel in the reactor. Following removal from the reactor, the irradiated fuels are generally stored under water for periods of from 90 to 120 days to permit decay of the short-lived fission products. The problems of handling and disposing of the product are intensified when shortened storage periods are employed.

After decay, the fuel material is processed. The processing varies depending on whether the fuels came from a heterogeneous or a homogeneous fuel reactor and on the materials used in the construction of the fuel element. The characteristics of a typical reactor fuel processing waste as reported by

Wolman and Gorman(1) are given in Table II. The wastes contain the fission products, some unrecovered fissionable material, and the various acids and salts required for dissolution of the material. These wastes, which are highly acid, may be stored in stainless steel tanks or they may be neutralized and stored in mild steel lined concrete tanks. Tank storage is the method presently employed at all atomic energy plants. Thus far we have had a little over a decade of experience with such tanks without any catastrophic results. However, the storage facilities have been located in relatively isolated or remote areas of the country. With the impetus being given to the use of nuclear energy for power development, it is reasonable to expect that nuclear power reactors will be located, and they are being located, close to large centers of population where greater safeguards may be necessary.

In the reactor types under consideration at present, particularly those employing solid fuels, chemical processing of the fuel was to take place at one of the AEC sites having such facilities. However, on January 5, 1956, the AEC advanced a policy aimed at having commercial chemical plants ready to process spent fuel elements from the first privately owned power reactors.(9) In addition the Commission would supply the plants with an initial base load of spent fuel from one or more of perhaps 20 reactors. Since these facilities will probably be near proposed reactor stations or at least near transportation facilities, it is probable that they too will be near centers of population. In the case of homogeneous reactor stations, the chemical processing plant, at least for the partial treatment of the fuel, will have to be located adjacent to the reactor. Under these conditions, the location of facilities for the storage of the large quantities of waste resulting from an expanding nuclear power economy is of great concern. Accordingly, considerable research and study is being given to the development of more satisfactory and permanent methods of disposal. Some of the proposed methods have been studied on a laboratory scale and are now being investigated in pilot plant facilities. Others are only in the thinking stage. The methods that have been demonstrated or suggested will be discussed in somewhat greater detail in the sections which follow.

Storage

Storage itself is not being abandoned even though the cost on a per gallon basis seems rather high. Prices quoted range from about 30 cents to $2.00 per gallon of tank capacity. However, when calculated on a kilowatt hour basis the cost is not high. Because power reactor wastes will have higher activity levels, it is necessary to provide facilities for cooling the waste solution. In conventional terms, the energy release in separations plant waste is small, up to 1 watt per liter of solution, but its management is complicated by the large volumes involved and non-uniform distribution promoted by the tendency to form precipitate in wastes normally made alkaline to minimize corrosion.(10)

Separation and Storage

Glueckauf(11) suggests that the quantitative removal of Sr^{90} and Cs^{137} from the wastes and separate storage from the bulk of the radioactivity will permit consideration of relatively short-time storage (about 20 years) for the residual activity. By such separation and storage it may be possible to recover the more usable radioisotopes and to release the residual activity under suitable conditions into either the ground or the oceans. A similar scheme is

under investigation at the Oak Ridge National Laboratory where the removal of specific radionuclides (strontium, cesium, zirconium, niobium, yttrium, ruthenium, and cerium) from simulated wastes by precipitation[12] and solvent extraction[13] has been evaluated. The separation of the waste into such components with separate storage of the more hazardous fraction has advantages in that less storage space will have to be provided, less elaborate storage may be provided for the bulk of the wastes, self-boiling of the wastes will not take place, and there will be no need to provide special tank cooling facilities.

Higgins and Wymer,[39] Chemical Technology Division, ORNL, proposed a scheme based on experimental work for decontaminating radioactive wastes whose chief ingredients are aluminum nitrate and nitric acid. Ninety per cent of the niobium and more than 95% of the ruthenium and zirconium are removed by a ferric hydroxide-manganese dioxide scavenging precipitation. The aluminum nitrate is converted to dibasic aluminum nitrate by destructive distillation and dissolved by 12-hour digestion at 160°C. The strontium, cesium, and rare earths are removed by cation exchange in a continuous contactor. The decontaminated waste can be volume-reduced to about 6 M aluminum before disposal and then stored cheaply. The fission products can be eluted from the column, concentrated into a small volume of highly radioactive waste, and stored with all necessary precautions.

Another scheme reported by Glueckauf and Healy[14] for the separation of cesium and strontium involves the following: The fission products are taken to dryness to remove all nitric acid and water; then the dry material is roasted for about an hour at 300°C., whereby all the nitrates except those of the alkali and alkaline earth metals are decomposed into oxides. The solid is then leached with warm water which dissolves out the cesium and strontium nitrates, leaving behind all the water insoluble oxides. Usually about 95 per cent of the cesium and 85 per cent of the strontium can be extracted. The roasting temperature is critical to $\pm$ 10°C., otherwise the leached material either contains too many impurities (at $<$ 290°C.), or retains too much cesium and strontium with the oxides (at $>$310°C.). The strontium and cesium recovered in this way may be further concentrated for separate storage and the residual fission products may be stored for a shorter time in separate facilities.

Fixation of Wastes and Firing

Ginell[23] and Hatch et al[24, 25, 26] have conducted experiments dealing with the fixation of radioactive contaminants on montmorillonite clays. In their process, the fission products were passed through a column containing extruded clay (spaghetti) and the clay was fired at temperatures up to 1000°C. Subsequent leaching showed that the amount of material leached was a function of the initial firing temperature. At temperatures approaching 1000°C. the amount of leaching was small and stabilized quickly. More recent investigations by the Brookhaven group have extended this method of disposal to high level wastes. Because of the large amounts of nitric acid or aluminum nitrate in the waste solutions, and because these quickly saturated the montmorillonite clay, it was necessary to pretreat the wastes to remove these high concentrations of stable materials. This was done originally through the use of permselective membranes. More recently, the pretreatment has been modified and now includes kiln-drying of the aluminum nitrate-fission products

waste to form aluminum oxide. The aluminum oxide plus fission products is leached; the leachate containing a fraction of the fission products is then passed through montmorillonite clay. The clay columns are fired and the activity fixed permanently.

The approach used by the Health Physics Division, Oak Ridge National Laboratory, was somewhat different in two respects. One, adsorption on the indigenous Conasauga shale was investigated, and two, fixation following mixing of the Conasauga shale and acid aluminum nitrate waste with specific quantities of limestone and sodium carbonate was evaluated. It was found that sintering at temperatures above approximately 500°C. could fix much of the activity. Subsequent leaching with water showed little release of activity with the exception of cesium. The next approach investigated the possibility of utilizing the heat of decay to fix the activity directly to the soil without the need of any external heat. The amounts of heat available have been reported by Perring[27] and could be sufficient, with enough concentrated activity, to permit self-fixation of the waste.

An initial laboratory investigation to determine the amount of heat necessary to fix such activity was undertaken by the Ceramics Group, Oak Ridge National Laboratory, and their studies led to the development of the so-called pilot plant hot-pot experiment which employed outside sources of heat. Fixation and sintering of the acid aluminum nitrate-fission products-flux mixture was effected and studies are now being planned in which high level wastes will be employed to determine the self-heating and self-fixing characteristics of such mixtures.

Amphlett[28,29] has investigated this problem in England and reports:[29]

"The use of natural silicate materials or soils as bases for formation of unleachable products, by mixing with the waste and firing to high temperatures, offers promise. Higher loadings can be achieved than by means of ion exchange, and inactive ions and acid do not appear to affect the adsorption of activity. This method may enable absorption of bulk wastes in one operation, and because the shorter-lived nuclides are included there is a possibility of self-fixation on a reasonable scale. No preparation of the material is required, and the process may be applicable to slurry and solid wastes as well as to solutions. If possible, the process should be aimed at melting the mixture to form a glass, in order to achieve the maximum density and hence the maximum concentration factor. This requirement is of course subject to leaching properties, loss of activity by volatilization during firing, and the possibility of lowering the melting point by the use of suitable additives. It is clear that much further work is required."

Mawson[30] refers to pilot plant studies made at Chalk River in which a 2.5 N nitric acid waste of high activity is mixed with nephaline syenite, and fused at a temperature slightly below 1000°C. forming an opal glass. Tests have shown that a very small amount of activity is leached initially, but after a short time very little activity comes off.

Ground Disposal

Many proposals have been made relative to the deposition of liquid wastes directly into the ground. These have been reviewed by several authors[31,32, 33, 34] and include disposal into shallow pits as has already been discussed, deep wells, cavernous rocks, unsaturated sediments, and saline deposits. The factors which must be evaluated in considering the feasibility of disposal of

high level radioactive wastes into the ground include: (1) the chemical and radiochemical content of the waste; (2) the effectiveness of retention of the radioisotopes in the available soil column above the ground water table; (3) the degree of permanence of such retention, as influenced by subsequent diffusion, leaching by natural forces, and additional disposal; (4) the natural rate and direction of movement of the ground water from the disposal site to public waterways, and possible changes in the characteristics from the overall liquid disposal practices; (5) feasibility of control of access to ground water in the affected region; (6) additional retention, if any, on sands and gravels in the expected ground water travel pattern; (7) dilution of the ground water upon entering public waters; (8) maximum permissible concentrations in those public waters of the radioelements concerned; (9) the temperature and pressure effects resulting from the heat of decay; and (10) the effect of waste discharges on present or potential mineral wealth of the region. High level wastes are not discharged into the ground in any location at the present time.

Ocean Disposal

As reported earlier, the only direct discharge of radioactive waste materials into the ocean is at Sellafield, England, where liquid wastes from the Windscale plant are discharged into the Irish Sea. In other instances, packaged materials are dumped in specified disposal areas, but in all cases these are of low or intermediate level wastes. There has been no discharge of high level radioactive materials. Some of the problems facing the oceanographer in determining the feasibility of such a disposal scheme have been reported by the British in describing their studies prior to the Irish Sea discharges.(35, 36,37) It is obvious that mixing, either desired or undesired, will play a major role in defining suitable disposal areas, if such exist.

With respect to ocean disposal of radioactive materials, the National Academy of Sciences report(38) states:

"Sea disposal of radioactive waste materials, if carried out in a limited, experimental, controlled fashion, can provide some of the information required to evaluate the possibilities of, and limitations on, this method of disposal. Very careful regulation and evaluation of such operations will, however, be required. We, therefore, recommend that a national agency, with adequate authority, financial support, and technical staff, regulate and maintain records of such disposal, and that continuing scientific and engineering studies be made of the resulting effects in the sea."

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TABLE I.

Waste Pit Characteristics

(as of October 31, 1955)

	Waste Pit Number:	1	2*	3
1.	Design capacity - gal	180,000	1,000,000	1,000,000
2.	Dimensions of pit - length, width, depth - ft	100x20x15	210x100x15	210x100x15
3.	Period of use	8/1/51-10/5/51	6/20/52-10/31/55	1/24/55-10/31/55
4.	Raw waste added - gal	123,000	1,331,760	1,434,600
5.	Waste from Pit 3 - gal		398,200[a]	-398,200
6.	Total volume waste - gal	123,000	1,729,960	1,036,400
7.	Beta activity added-curies	389	15,975	18,911
8.	Beta activity taken from Pit 3 - curies		3,449[a]	-3,449
9.	Total beta activity - curies	389	19,424	15,462
10.	Radioisotopes present Per cent of total			
	Cs^{137} - Ba^{137}	60	60*	77
	Ru^{106} - Rh^{106}	40	40*	12
	Sr^{90} - Y^{90}			11
11.	pH of waste added	~ 12.5	~12.5	~10-11
12.	Approximate construction cost - dollars/million gal	$14,500	$14,500	$14,500
13.	Approximate cost/gal waste capacity - cents	1.45	1.45	1.45
14.	Approximate cost-gal waste discharge to pit - cents	2.12	< 0.84[b]	< 1.40[b]

* Decanted liquid from Pit No. 3 and added the material to Pit No. 2 after 1/24/55.

[a] Volume of waste and activity decanted from Pit No. 3 and added to Pit No. 2. Activity estimated from activity added just before decanting.

[b] Based on total gal waste volume in pit. Equal to line 13 values divided by line 6/1,000,000 values.

TABLE II.

Characteristics of Typical Reactor Fuel Processing Wastes

(High-level waste)

Gross beta activity, $1.6 \times 10^6 - 2.2 \times 10^{10}$ cpm/ml
Alpha activity, $6.0 \times 10^3 - 6.0 \times 10^5$ cpm/ml
Radioactivity, 1 to 4 x 10^2 curies per gallon (neutralized)
Effective life, about 600 years
Heat generation, 1 to 3 BTU/hr/gallon
Power equivalent, 1 gm U^{235} = 24,000 kwh (100% efficiency)
Fission product wastes, 1 gm U^{235} forms 1 gm fission products
Wastes from processing 0.5 to 5.0 gal waste solution/gm U^{235} consumed

Waste chemistry:

Ions	Concentration in moles per liter
Al	0.5 - 2.5
NO_3	2.0 - 8.0
H	0.5 - 3.0
Na	0.1 - 0.2
F	2.0 - 3.0
Zr	0.3 - 0.6
SO_4	0.3 - 0.6

Specific gravity, 1.1 to 1.4

Paper 1276 SA 3

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

AN ANALYSIS OF SAND FILTRATION

Warren A. Hall*
(Proc. Paper 1276)

INTRODUCTION

Successful filters utilizing sand as a means of removing most of the suspended matter and bacteria from water supplies have been used at various times for more than forty centuries.(1) In spite of this, the nature of the process by which solids are removed by the filter has not been clearly defined. The history of sand filtration tells of a succession of filters, each more or less satisfactory, from which a set of design rules has been derived. While filters will continue to be improved by this trial and error process it has become apparent that a better knowledge of the fundamental mechanism of filtration must be obtained.

The American Society of Civil Engineers, through the Sanitary Engineering Division, instituted in 1935 an experimental study of the effects of differently composed filter sands on the operation of the rapid sand filter.(2) Baylis(3) has reported on increased infiltration rates made possible by improving the strength characteristics of the floe. The success of these experiments indicate that with a satisfactory theory of the mechanics of filtration for a basis, additional experimental advances may be possible. With this end in view an attempt is made in this paper to develop a relationship for the distribution of sediment in a filter in terms of space time coordinates and the physical properties of the components of the system.

DEVELOPMENT OF THE FUNDAMENTAL EQUATION

The limited data available on sediment distribution in the filter after a run appear to substantiate a postulation that only a portion of the suspended matter is removed at any given depth.(2) There is also a possibility that a "creep" or low velocity flow of the semi-fluid deposited sediment through the filter may occur. Both possibilities are accepted in the preliminary analysis. A second postulation requires that the reduction in concentration of suspended matter may be considered to be a continuous process. Utilizing the principle of mass conservation for one dimensional flow, an equation of continuity for the sediment will be derived. The weight rate of inflow of solids G_i to an

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elemental volume Adz is the sum of the weight of sediment carried as suspension and that due to "creep" flow. Let C be the concentration of suspended matter, in weight per unit volume, v be the rate of water flow through the filter, distance per unit time, v_c be the creep flow rate, distance per unit time and γ_c be the specific weight of the creep flow sediment. Then the weight rate of inflow is

$$G_i = vCA\,dz + \gamma_c v_c A\,dz \tag{1}$$

The weight rate of outflow is

$$G_o = [vCA + \partial(vCA)/\partial z]dz + [\gamma_c v_c A + \partial(\gamma_c v_c A)/\partial z]\,dz \tag{2}$$

The rate of accumulation of sediment per unit total filter volume per unit time is equal to the net inflow over outflow divided by the element of volume. If W is the weight of sediment accumulated per unit filter volume at any point then:

$$\partial W/\partial t = -v\,\partial c/\partial z - \partial(\gamma_c v_c)/\partial z \tag{3}$$

This equation may be termed the fundamental differential equation of filtration. The second term on the right hand side is most probably a non linear function of the deposited sediment. Its analysis must await development of suitable methods to study flows of this type. Fortunately in many instances creep flow appears to be negligible, for example, during the initial portion of a run. This term will be omitted in the analysis to follow.

Immediately following the start of a filter run the filter may be considered to be a homogeneous isotropic porous medium. Thus the mechanism of separation, whatever its nature, should be the same throughout the filter. The distribution of suspended particles of a given size in a volume of the order of the dimensions of a pore is most probably uniform. From these conditions it follows that the rate of change of concentration of suspended matter of a given size is a function of the concentration of that size in suspension. Further, if each individual particle of the suspension behaves independently of the others, this function will be a simple proportionality. Designate the factor of proportionality by r, hereafter termed the rate factor. The rate factor is thus seen to vary with particle size but not with position in the filter so long as the conditions of homogeneity and isotropy are approximated. For any point in the filter at some instant of time

$$dC/dz = -rC \tag{4}$$

Separating the variables and integrating between the limits C_0 and C, O and z:

$$C = C_o e^{-rz} \tag{5}$$

C_0 represents the initial concentration of suspended matter in the water as it enters the filter and z is the depth below the surface of the filter. Taking the partial derivative of C with respect to z gives:

$$\partial C/\partial z = -rC_o e^{-rz} \tag{6}$$

Substituting equation 6 in equation (3) we have, neglecting the creep flow term:

$$dW/dt = vrC_o e^{-rz} \tag{7}$$

An equivalent result which will aid in interpreting the rate factor may be derived for a discontinuous filtration process, i.e., one in which the separation occurs only at certain points in the filter such as the front face of a sand grain or the narrow crevice at points of contact between sand grains. A new rate factor r' is defined to be the fraction of the concentration C removed in a distance equal to the center to center distance between sand grains. Designate this distance by h, then the relationship between r and r' may be expressed in terms of h.

$$r' = rh \tag{8}$$

In the first layer of sand the concentration will be reduced by an amount $r'C_o$. Thus the concentration leaving this layer and subsequent layers will be:

$$\begin{aligned} C_1 &= (1-r')C_o \\ C_2 &= (1-r')C_1 \\ C_i &= (1-r')C_{i-1} \end{aligned} \tag{9}$$

Substituting and summing to obtain the concentration after the ith layer gives:

$$C_i = C_o(1-r')^i \tag{10}$$

The rate of accumulation of sediment at any point in the filter is equal to the rate of reduction of concentration of suspension at that point. Hence

$$dW/dt = vr'C_o(1-r')^{z/h} \qquad z/h = 1,2,3,\cdots i \tag{11}$$

As h decreases relative to z this expression approaches the value given by equation (7).

Analysis of the Rate Factor

The development in the preceding section considers the continuity requirements but not the effect of the other physical properties of the system. These properties are embodied in the value of the rate factor. In order to estimate how these properties might affect the rate factor they will be reviewed very briefly.

The hydraulics of the flow through the sand is extremely complex. However, certain comments may be made. Experience with channel flow shows that in channels where the fluid is accelerating, velocity profiles tend to become more uniform and have less tendency to develop eddies. The opposite is true of a decelerating flow.[4] Hence we may presume that the flow in a

pore approaching a constriction over the front face of a sand grain has a tendency toward more uniform velocity profiles, while the flow away from the constriction will tend to accentuate differences of the velocity profile and to form eddies when flow rates are high enough.

Most discussions of flow in porous media consider as a criterion for turbulence a Reynolds number of unity based on a means and diameter and a velocity equal to the discharge per total area of filter. Since many filters operate in or near the range $1 < N_R < 10$ this matter requires some discussion. The flow of a fluid about an isolated sphere may give some insight to the flow very close to the sand grain. At a Reynolds number of unity, based on the same data as used for the criterion in a porous medium, a separation of the laminar boundary layer occurs resulting in the formation of two small eddies behind the sphere. As the velocity is increased these eddies grow in size until they begin to be shed from the sphere. Subsequently the true turbulent boundary layer develops. Applying these facts to a porous medium is of course difficult. However, it is justifiable to expect that a similar progression of the nature of the flow in the boundary layer would occur although possibly at different values of N_R due to the presence of other sand grains and the effect of the non-spherical shape of the grains. Thus for porous media, the departure from a straight line of the plot of the log of the loss coefficient vs. the log of the Reynolds number which occurs at a Reynolds number of unity is probably due more to the formation of localized eddies behind the sand grains than to the incidence of self propagating turbulent (mixing) flow. These eddies, by dissipating energy and by increasing the actual velocity due to constriction of the areas, are probably responsible for the variation observed in the loss coefficient curves in the range $1 < N_R < 10$.

The floc in the water reaching the filter has passed through a coagulation, mixing and settling basin. Applying Stokes law to the settlement of particles, it is obvious that particles larger than the largest size which would settle in the time the water passes through the settling basin should not reach the filter. On the other hand if proper mixing and coagulation has occurred there should be few particles remaining in the near colloidal range. Thus particle sizes may be expected to lie in a relatively small range. The floc appears to possess cohesiveness or strength. It probably possesses adhesiveness with respect to the sand of the filter and to other particles.

The sand in the matrix of the filter may be considered to be spherical for most purposes of analysis with deviations in shape accounted for by correction factors. Although filters usually are not constructed with uniform sands, the results of the investigations of the American Society of Civil Engineers, Sanitary Engineering Division(2) indicate that uniform sands are as good or better than graded sands. Grading does not appear to be a necessary feature of a filter and sand size may be presumed constant for the theoretical filter.

The mechanism of filtration should be compatible with the foregoing remarks concerning velocity profiles, lack of fully developed turbulence, narrow range of particle sizes and floc properties. One possible mechanism to be considered postulates that an action occurs in the crevices adjacent to points of contact of the sand grains. In each of these angular areas normal to the flow there is a width of opening smaller than the geometric size of any given suspended particle. Whatever portion of the flow, however small, passes inside this point will be stripped of all particles of that size or larger. Thus the rate of removal at each sand layer for a given particle size is determined by the ratio of the flow passing within such boundaries to the total flow. A

second compatible mechanism visualizes the small voids as settling basins in which gravitational sedimentation occurs assisted slightly by a dynamic effect due to the difference in densities of floc and water. It should be noted that if both straining and gravitational mechanisms are active, their results are not additive, since each removes only certain of those particles in close proximity to the surface of the sand grains.

The Interstitial Straining Theory

Figure 1 represents an idealized cross section through a constriction. If the area between the arc segments is presumed to be approximately the same as that between the chords, the area of the constriction with a dimension smaller than the particle size D, may be determined from geometric considerations to be approximately

$$A' = (3/2\sqrt{2})\, D^{3/2} d^{1/2} \tag{12}$$

Where D is the diameter of the particle and d is the diameter of the sand grain, the total area of the constriction is:

$$A = 0.303\, d^2 \tag{13}$$

Introducing the constant c to include the effects of variation in velocity profile, non-symmetry of the particles and other grain arrangements than that used above, the rate factor for discontinuous separation is given approximately by the ratio of the two areas multiplied by the correction (experimental) constant.

$$r' = 35c\, (D/d)^{3/2} \tag{14}$$

Subsequent comparison of experimental data and the theoretical values indicates a value for c of 0.1 should be of the proper order of magnitude. Considering the effect of the converging channel on the velocity profile, this value appears quite reasonable.

The Gravitational Sedimentation Theory

Consider next the flow of a suspension of uniform size particles past an array of spheres. The direction of flow is parallel to the acceleration of gravity. If the particle had the same density as the water, the velocity and path of the particle would be identical to the velocity and a streamline of the fluid. Since the particle has a greater density than water, an unbalanced force will exist initially and the particle will accelerate relative to the fluid until a particle velocity relative to the fluid is reached for which the viscous drag is equal to the unbalanced gravitational force. A second unbalanced force exists due to the curvatures in the streamlines of the fluid in passing around a sphere. On the upstream face this acceleration tends to cast the particle nearer the sphere. The two unbalanced forces are directly proportional to the corresponding accelerations and the opposing viscous forces are

proportional to the corresponding relative velocity components. Therefore, the ratio of the relative velocities is equal to the ratio of the accelerations. Since for the conditions encountered in sand filters the acceleration of gravity is very much greater than the acceleration due to curvatures, the relative displacement of a suspended particle due to flow curvatures may be neglected. The suspended particle may thus be considered to be moving downward everywhere in the filter with a velocity relative to the fluid determined by Stokes law. Let the specific weight of the particle be γ, the specific weight of the fluid to be γ_0 and the viscosity of the fluid be μ. The velocity of the particle relative to the fluid is:

$$V = 2/9 \, (\gamma - \gamma_0) D^2/\mu \qquad (15)$$

For convenience the coefficient of D^2 will be given by a single coefficient m in the subsequent analysis. Since V is the velocity of the particle relative to the fluid anywhere in the fluid it is also equal to the vertical velocity at which particles strike the boundaries of the fluid, i.e. the sand grains. Presume the particles do not reenter the fluid because of their adhesiveness to the sand. The rate at which particles are brought into the region h is vC and the rate at which they settle on the sand grains is VC. Hence the fraction removed in the region h is the ratio of the relative velocity V of the particles to the mean velocity of flow v with a correction constant as before caused by non-uniform velocity distribution in the pores and the ratio of solid area to void area at any crops section through the filter. The discontinuous rate factor for the gravitational theory is therefore:

$$r' = c' m D^2/v \qquad (16)$$

Experimental data indicate a value for c' of 0.1, which appears to be reasonable.

Experimental Verification of the Theory

The theoretical relationships to be verified are:

(a) $dW/dt = vrC_o e^{-rz}$

or (b) $dW/dt = vr'C_o (1-r')^{z/h}$

(c) $r' = 35_c (D/d)^{3/2}$ (straining theory)

and/or (d) $r' = c' m D^2/v$ (gravitational theory)

The relationship between r' and r is

(e) $r' = rh$

These theoretical relationships may be compared with data given by the Committee of the Sanitary Engineering Division on Filtering Materials(2) in figure 8 of their report. The results are given there in terms of "accumulated turbidity," that is, the sum of all material caught above a given depth. Since graphical differentiation of the data is somewhat inaccurate, the theoretical relationships will be integrated for comparison purposes. Integration of (a) with respect to time gives the total accumulation per unit volume at a point.

$$W = vrC_o te^{-rz} \tag{17}$$

Integrating this expression with respect to z from O to z gives the total weight of accumulated turbidity T per unit filter area which lies above a depth z.

$$T = vC_o t\,(1-e^{-rz}) \tag{18}$$

It may be noted that the coefficient qC_0t represents the total sediment to be removed during the time interval t, hence, the percentage of total sediment which is accumulated above a point z is

$$P = 1-e^{-rz} \tag{19}$$

Substituting the discontinuous rate factor r', presuming $z >> h$

$$P = 1-e^{-r'z/h} \tag{20}$$

Although the data presented by the Committee(2) did not indicate particle size of floc entering the filter some information is given concerning the presedimentation basin from which approximate particle sizes were estimated as lying between 10 and 25 microns. Accordingly theoretical curves are plotted as solid lines in figure 2 for 10, 12, 15, 20 and 25 micron particle sizes for comparison purposes. The curves are plotted using rate factors derived for the gravitational straining theory. Quite by coincidence the rate factor for each of the two filtration mechanisms studies are approximately equal under the experimental conditions of sand size, particle size, and flow rates. Both theories thus seem to be acceptable. From the nature of the equations it would be expected that straining would predominate as the flow rate increases or the sand size decreases while the gravitational settling would predominate as the sand size increased or the flow rate decreases. It should also be noted that the two mechanisms are not additive in their effects since both remove particles nearest the sand surface.

The experimental data are replotted on figure 2 from the ASCE report in terms of percent of total accumulated sediment by presuming the maximum turbidity removed represents one hundred percent. There are three general groups of curves given in figure 8 of the report. Individual points from runs number 6 and 7 are replotted in figure 2 and are both seen to lie close to the theoretical curve for 15 microns. Runs number 2, 4, and 5 are also essentially congruent and lie close to the theoretical curve for 12 microns. Runs 1 and 3 are difficult to compare as it would appear that, if the filter had been

deeper, a higher filtration efficiency would have been attained. If it is presumed that a deeper filter would have given approximately 50 ppm more total accumulated turbidity for run number 3, as is indicated by the shape of the other experimental curves, then run number 3 will lie very close to the curve for 10 microns. Run number 1 is not plotted since there is no indication of a break in the curve with which to estimate the probable initial total turbidity in the applied water.

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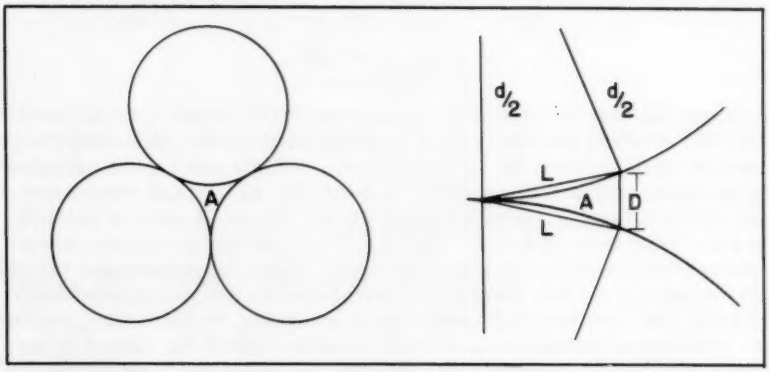

Fig. 1. Idealized Geometry of a Cross Section of a Pore Space.

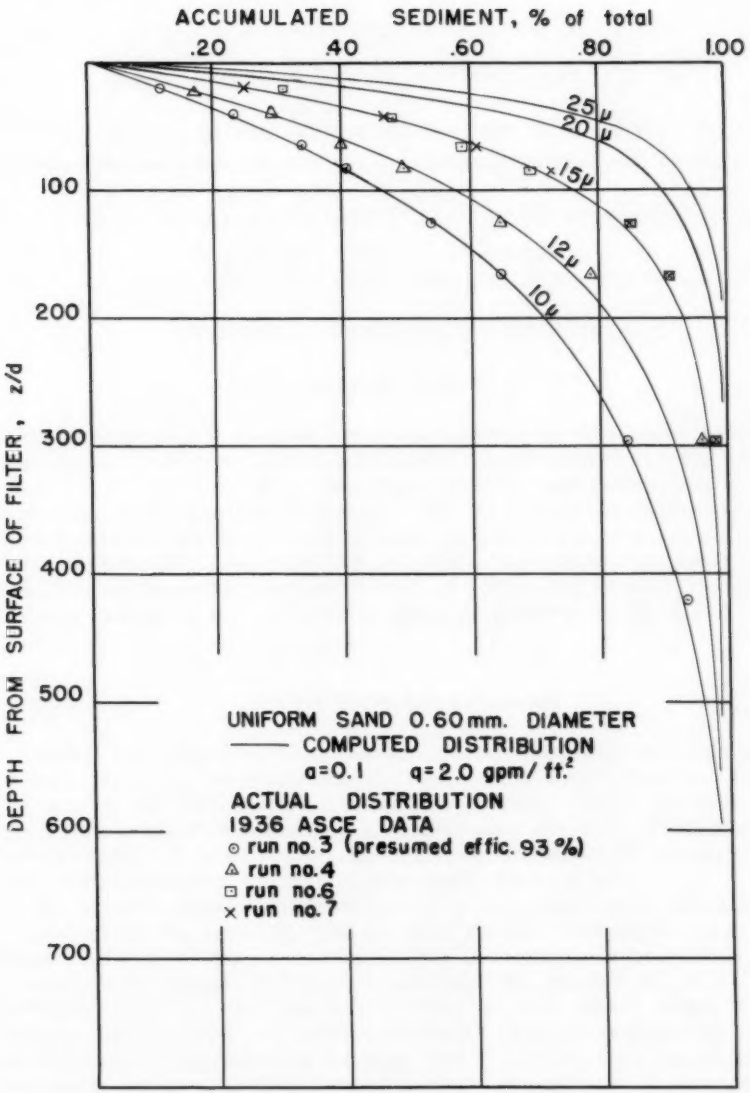

Fig. 2. Sediment Distribution as a Function of Depth of Filter. Solid Lines are Based on the Derived Theory. Individual Points are Based on Data Published by the A.S.C.E. Committee on Sanitary Engineering.(2)

Paper 1282 SA 3

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SALARIES OF LOCAL ENVIRONMENTAL HEALTH PERSONNEL IN 1956

Report of the Committee on Salaries
Conference of Municipal Public Health Engineers

SECTION I - THE GENERAL PICTURE

The Purpose of the Study

The purpose of this study is to present information on salaries paid to professional workers in environmental sanitation employed by local health departments. In collecting the data, the Committee has related salary levels of environmental sanitation positions to such factors as population served, educational and experience requirements, as well as supervisory and program responsibility. An effort has been made to establish relationships with economic and geographic factors as well as with salary levels of related professional groups, in order to provide some perspective for the findings of the study.

How the Study was Conducted

During June, 1956 the Committee sent questionnaires to 1492 local health departments in the United States and Possessions. Usable replies were received from 371 local health departments representing 3,251 professional environmenta sanitation positions. It is estimated that there are 7500 such workers today. In other words, this study includes about 43 percent of all environmental sanitation workers in local health departments.

Two types of questionnaires were sent to each department. Sample copies are included in the appendix. One provided general information covering the department and the other provided detailed information covering each class of professional positions. A "class of positions" was defined as: "A group of positions which have the same payroll title, the same salary range, the same entrance requirements and similar job duties. Where examinations are given, all applicants for positions in the same 'class' will be given similar tests." Data were received for 1019 separate classes of positions. Each questionnaire was edited and coded according to a predetermined code. Each salary was coded for the $500 bracket in which it occurs. The editing was designed to make as many returns as possible usable without affecting the basic data. The edited and coded questionnaires were punched and machine tabulated by

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The Office of Statistics and Research of the Philadelphia Department of Public Health.

During 1951(1) and 1954(2) the Conference carried out salary studies on a group of about 35 departments. In order to provide chronological information on salary levels and related data, the same 35 departments were included in this year's study and special tabulations from this group are included in this report.

How the Committee will Present its Findings

Because of the large volume of data collected in the study, the Committee proposes to present its Report in the form of five sections as follows:

Section I - The General Picture
Section II - Engineers
Section III - Sanitarians
Section IV - Other Sanitation Personnel
Section V - Conclusions and Recommendations

What was Surveyed?

The study included 371 full-time local health departments in the United States, Puerto Rico, Hawaii and the Virgin Islands. The population served by these departments is shown in Table I. In addition this table shows the population distribution of the sample of 35 departments which were included in salary studies in 1951 and 1954 and will be used in this study for comparison. It is evident that the larger departments are represented in greater proportion in this sample.

How Much did Environmental Sanitation Positions Pay in 1956?

Since most government positions are covered by a salary range, and since such personnel are usually in various steps in such a range, it was felt that valid comparisons of positions would have to be based on either the minimum or the maximum salary of the range. Most of the comparisons in this study are based upon the maximum salary of the range.

Chart I shows the distribution of the median maximum (statistical and other terms are explained in Appendix 1.) of the salary range of each professional category. There are considerable differences between these professional categories with respect to the range of maximum salary levels. This is particularly true for sanitarians, engineers and veterinarians where maximum salary levels vary considerably with job responsibility and educational and experience requirements. These factors and their relation to salaries will be discussed in later Sections of the Committee's Report.

Median maximum salaries for directors of environmental sanitation programs are shown in Table 2.

Salaries paid to sanitarians and lay health officer--sanitation directors are considerably below those paid to other professional categories holding director positions. This is partially due to the fact that this group usually holds director positions in smaller departments.

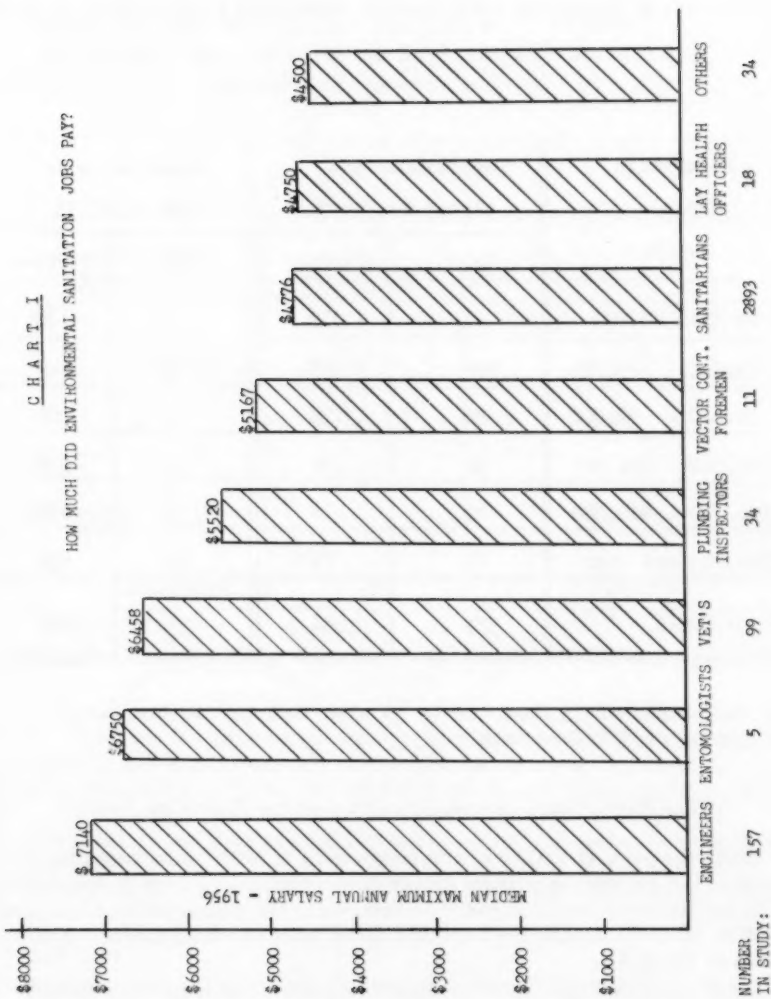

C H A R T I
HOW MUCH DID ENVIRONMENTAL SANITATION JOBS PAY?
MEDIAN MAXIMUM ANNUAL SALARY - 1956
$8000
$7000
$6000
$5000
$4000
$3000
$2000
$1000
$7140
$6750
$6458
$5520
$5167
$4776
$4750
$4500
ENGINEERS
ENTOMOLOGISTS
VET'S
PLUMBING INSPECTORS
VECTOR CONT. FOREMEN
SANITARIANS
LAY HEALTH OFFICERS
OTHERS
NUMBER IN STUDY:
157
5
99
34
11
2893
18
34

TABLE 1

Population served by Local Health Departments included in the study.

Population Range	Departments included in Study		Departments in sample of 35	
	No.	Percent	No.	Percent
Less than 25,000	61	17%	1	3%
25,000 - 49,000	107	29%	0	-
50,000 - 99,000	86	23%	2	6%
100,000 - 199,000	50	13%	8	23%
200,000 - 499,000	42	11%	9	26%
500,000 and over	25	7%	15	43%
TOTAL	371	100%	35[1)]	100%

1) The sample of 35 Departments is also included in the larger group of 371 Departments.

How Have Salary Levels Changed over the Past Five Years?

Using the present salary of commissioners of health as a basis for comparison, the average maximum salary of sanitation directors in our sample of 35 departments, had increased in relation to the commissioner's salary up to 1954. Since that year the ratio has fallen off to a small extent. This is shown in Table 3.

The Municipal Public Health Engineering Report to the Engineering Section appearing in the 1947 issue of the APHA Journal (page 903) stated that: "The engineer-director should ordinarily command the second highest salary in the department. . . ." It further suggested that the salary range of the engineering director be 65% to 80% of that of the health officer.

The percent of departments which have personnel other than the commissioner with salaries equal to or higher than that of the director of sanitation has risen from 65% in 1954 to 72% in 1956. Among the departments which have personnel with salaries higher than the sanitation director, the average number of such personnel has dropped from 3.9 in 1954 to 3.1 in 1956. Data concerning such personnel in relation to population served by the department is shown in Table 4.

TABLE 2

Median Maximum Salaries of Sanitation Directors by Professional Category in 1956 for 279 Local Health Departments.

Professional Category	No. Reporting	Median Maximum Salary
Sanitarian	196	4,878
Engineer	67	8,141
Lay Health Officer	13	5,250
Veterinarian	3	8,960[1)]

1) Average - too few respondents to calculate median.

Salary data over the 1951-56 period for directors of environmental sanitation, engineers and sanitarians are shown in detail in Tables 5, 6 and 7. These data were obtained from the sample of 35 or more local health departments which have been used by the Conference in previous studies.(1,2) Chart II presents these data graphically together with comparative data from other sources.

On reviewing this Chart certain facts became apparent; they are as follows:

1. Salaries have been rising steadily at rates varying from 5 to 13 percent each year.
2. The average top salary paid to an engineer in 35 local health departments was in 1952-54 period $500 or more below the median salary for engineers(3) generally.
3. The median salary for engineers in local health departments was in 1952(3) and 1954(4) $1000 below the median salary paid to engineers in County and Municipal Governments(5) and $2400 below the median for all engineers.
4. The median salary for sanitarians in local health departments has been, since 1952(3) $2000 dollars or more below that of engineers in local

TABLE 3

Comparison of Health Commissioner's Present Salary

with Maximum Salary of Director of Environmental Sanitation

for 35 Local Health Departments

Year	Departments Reporting	Average Present Commissioner's Salary	Percent Salary Increase		Avg. Max. Salary of San. Dir.[1] as % of Commissioner's Salary
			Commissioner	Sanitation Director	
1951	34	10,092	20%	21%	62.7%
1954	35	12,127			68.3%
1956	35	13,464	11%	10%	62.6%

1) 90% of the directors in sample are engineers.

TABLE 4

Number of Health Department Personnel Receiving Salary Equal to or Greater than Sanitation Director in 35 Local Health Departments Grouped by Population Served: 1951, 1954, 1956.

Population Served	Year	Dept's. Reporting	Personnel having Salaries Equal to or Greater than Sanitation Director other than Commissioner			
			Departments having Such Personnel		Number of Such Personnel	
			No.	%	Range	Average
Less than 100,000	1951	3	1	33%	1	1
	1954	3	0	---	0	-
	1956	3	1	33%	1	1
100,000 - 199,000	1951	11	7	64%	1 - 3	-
	1954	8	3	37%	1 - 5	3.6
	1956	8	3	37%	1 - 4	2.0
200,000 - 499,000	1951	14	9	64%	1 - 9	3.6
	1954	10	6	60%	1 - 6	3.8
	1956	9	6	75%	1 - 4	2.3
500,000 - and over	1951	11	7	64%	1 - 20	9.2
	1954	14	14	100%	1 - 11	4.0
	1956	15	15	100%	1 - 7	3.8
All Departments	1951	38	24	63%	1 - 20	4.7 (Est.)
	1954	35	23	65%	1 - 11	3.9
	1956	35	25	71%	1 - 17	3.1

TABLE 5

Salary Levels of Sanitation Directors in 35 Local Health Departments

Grouped by Population Served: 1951, 1954, 1956.

Population Served		Less than 100,000			100,000 199,000			200,000 499,000			500,000 and over			All Departments		
Year		'51	'54	'56	'51	'54	'56	'51	'54	'56	'51	'54	'56	'51	'54	'56
No. Dept's. Reporting		3	3	3	13	8	8	13	10	9	10	14	15	39	35	35
Salary	Minimum[1]	3,864	3,600	4,250	4,620	5,000	6,250	3,900	5,810	6,750	4,850	5,760	7,250	3,864	3,600	4,250
	Maximum[1]	6,500	5,100	6,250	7,200	10,400	9,250	8,308	10,000	9,750	10,000	11,723	12,250	10,000	11,723	12,250
	Average Maximum	5,527	4,700	6,250	6,172	7,325	8,107	6,895	8,328	8,679	8,069	9,579	9,967	6,849	8,288	9,108
	Average Present	5,335	4,656	4,920	5,752	6,765	7,625	6,556	7,730	8,528	7,758	8,915	9,383	6,500	7,732	8,379
	Percent Change[2]	-13%		+6%	+18%		+13%	+18%		+12%	+15%		+5%	+19%		+9%

1) See appendix 1 for definition of terms. 1956 salaries are reported as the median of the $500 range in which they occur. (This also applies to Tables 6 and 7.)

2) Change in average present salary.

TABLE 6

Engineering Salaries, including Director (if Director is Engineer)
for 35 Local Health Departments Grouped by Population Served: 1954, 1956

Population Served		Less than 100,000		100,000 199,000		200,000 499,000		500,000 and over		All Departments	
Year		1954	1956	1954	1956	1954	1956	1954	1956	1954	1956
No. Depts. Reporting		3	3	8	8	10	9	14	15	35	35
No. Depts. with Engrs.		2	2	8	6	9	6	14	14	33	30
Average No. of Engrs.[1]		1	1	1.38	1.83	2.11	3.00	4.36	5.00	2.94	3.67
Salary	Lowest in Dept.	4,246	3,750	(3,600)[2] 4,200	3,250	(2,600) 3,520	3,750	(3,150) 4,108	4,250	(2,600) 3,518	3,250
	Top in Dept.	5,100	6,250	10,400	9,250	9,360	9,750	11,723	12,250	11,723	12,250
	Average Top	4,950	5,500	7,277	8,250	7,820	8,300	8,817	9,821	7,937	8,817
	% Change	+11%		+13%		+6%		+11%		+11%	

Note: 1) Based on departments with engineers
2) Data in parentheses are for 1951

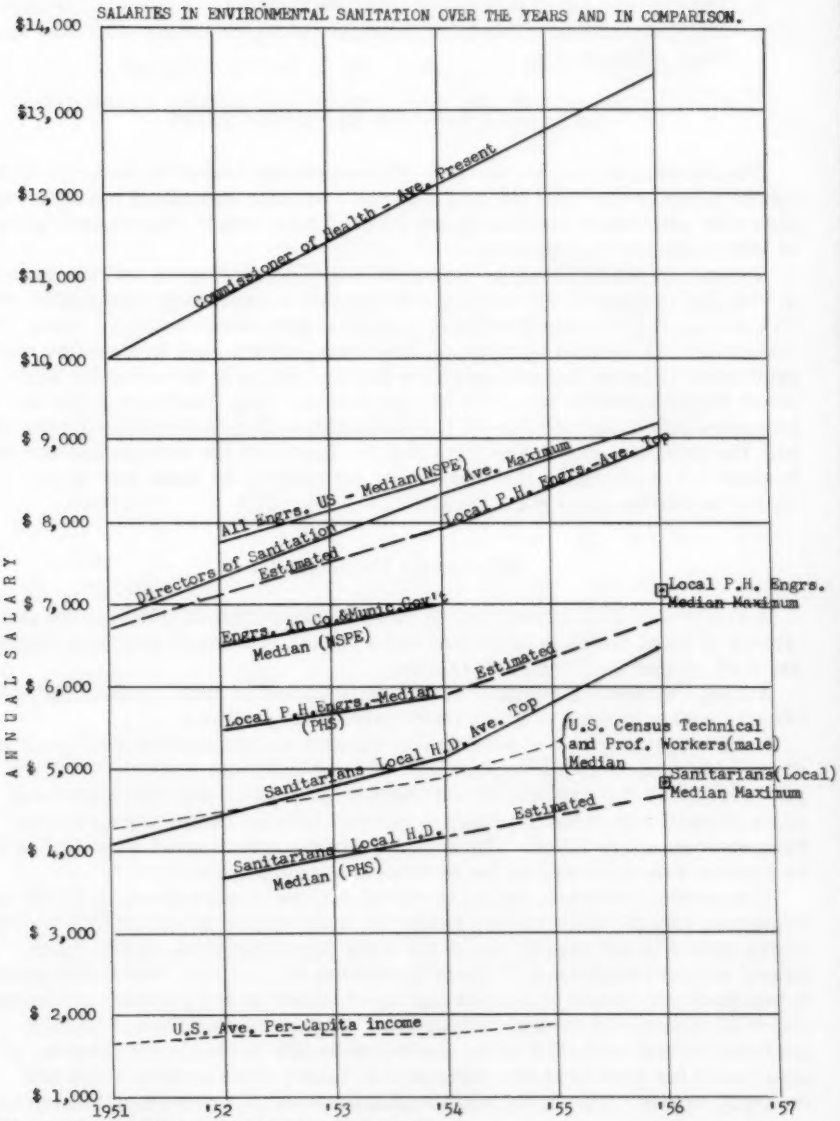
TABLE 7

Salaries of Sanitarians in 35 Local Health Departments
Grouped by Population Served: 1951, 1954, 1956

Population Served	Less than 100,000			100,000 199,000			200,000 499,000			500,000 and over			All Departments		
Year	'51	'54	'56	'51	'54	'56	'51	'54	'56	'51	'54	'56	'51	'54	'56
No. Depts. Reporting	2	3	3	12	8	8	14	10	9	5	14	15	33	35	35
Lowest in Dept.	2,400	2,630	2,250	2,100	2,628	3,250	2,100	2,820	2,750	2,500	2,520	2,750	2,100	2,520	2,250
Top in Dept.	4,000	4,200	4,750	5,300	4,764	7,250	5,748	7,800	7,780	4,044	9,770	11,750	5,748	9,770	11,750
Average Top	3,560	3,792	4,083	3,943	4,290	5,750	4,236	5,223	6,028	3,751	5,784	7,683	4,015	5,112	6,507
Percent Change	+7%	+8%		+9%	+34%		+23%	+15%		+54%	+33%		+27%	+27%	

CHART II

SALARIES IN ENVIRONMENTAL SANITATION OVER THE YEARS AND IN COMPARISON.



See Appendix 1. for explanation of abbreviations and terms.

health departments and approximately $750 below the median salary paid to male professional and technical workers in the U. S.(6)

5. The average top salary paid to a sanitarian in the sample of 35 local health departments has been rising more rapidly since 1954 than any other salary group.

Do Salaries Vary with Geographic Area?

The answer, of course, is—yes. Environmental sanitation salaries in local health departments vary not only with the economic makeup of the area, but also with patterns of organizing and staffing local health departments peculiar to and traditional for the area.

Table 8 shows the median maximum salary for engineers and sanitarians in various sections of the country. These data suggest that salary differences as great as 30% can be expected due to geographic factors alone. Engineering salaries in the various sections of the country do not rank in the same way as sanitarian salaries, for example New England ranks in 6th place for engineers as compared to 3rd place for sanitarians. The Southwest ranks 2nd for engineers as compared with 4th for sanitarians. It is interesting to note that, with the exception of the Far West and the Southeast for both groups and New England for sanitarians, the salaries do not rank in the same way as per capita income by geographic area.

What About Vacancies?

A study(7) in 1951 showed that 5.7% of all the budgeted positions for sanitarians in local health departments were vacant; this study indicates that today 6.8% of such positions are vacant.

Among engineering positions 15.9% were vacant in 1951. Today, 23% are vacant—nearly one out of every four engineering positions.

Data concerning vacant positions is difficult to interpret because policies differ with respect to the disposition of vacant budgeted positions. Vacant positions alone are rarely a direct index of demand or need for personnel since planned expansion of program is frequently postponed when existing vacancies cannot be filled. The actual demand for personnel, therefore may be greater than indicated by the number of vacant positions.

The median maximum salary of vacant engineering positions is $6,188 as compared with $7,140 for filled positions, a difference of nearly $1000. While a high salary is not usually one of the most important work satisfactions among sanitary engineers;(8) there appears to be a salary level below which it becomes impossible to recruit engineers under existing market conditions. Chart III is an effort to demonstrate this. It shows a variation of percent positions vacant from 45% in the Southwest to 14% in the Central States. It also shows for each area the difference in salary level between filled and vacant positions. The direct relationship between this difference and percent vacant positions in the various sections of the country is striking. The only exception is the Far West where salary levels both filled and vacant are significantly higher than in all other sectors. The median maximum salary for vacant positions in the Far West is $7,750. These data, together with the fact that the percent of vacant positions is increasing, suggests that drastic upward

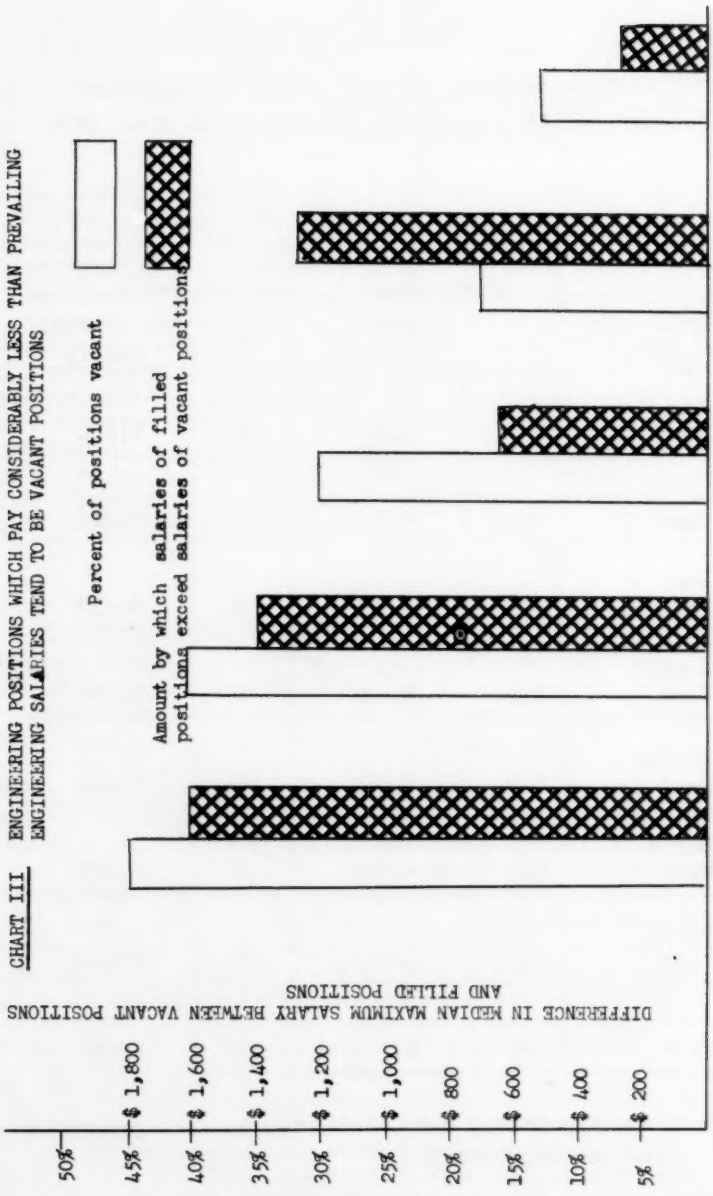
TABLE 8

Median Maximum Salaries of Sanitarians and Engineers in 371 Local Health Departments by Geographic Area, 1956

Geographic Area[1)]	Sanitarians		Engineers	
	No.	Median Max. Salary	No.	Median Max. Salary
Far West	646	5,614	12	9,000
Central	739	4,870	52	7,500
New England	78	4,866	5	6,750
South West	221	4,631	11	7,834
Middle East	467	4,630	47	7,250
North West	189	4,449	9	7,000
South East	553	3,917	21	6,725
U. S.	2893	4,776	157	7,140

1) Note: For States and Territories in each geographic area see Appendix 2.

CHART III ENGINEERING POSITIONS WHICH PAY CONSIDERABLY LESS THAN PREVAILING ENGINEERING SALARIES TEND TO BE VACANT POSITIONS



revision of salary levels among a significant proportion of engineering positions in local health agencies is indicated if these departments are to compete successfully for competent engineering personnel.

Among sanitarians there are also geographical variations with respect to vacancies. They are as follows:

Geographic Area	U.S.	Far West	North West	New England	Middle East	South East	Central	South West
% Positions Vacant	6.8%	9.9%	9.5%	8.9%	8.1%	6.5%	4.2%	2%

While the percent vacancies among sanitarian positions is considerably smaller than for engineering positions, an analysis of salary data for the vacant positions produces interesting results. In the southeastern, southwestern, northwestern and New England States, median maximum salaries of vacant positions are actually from 5% to 12% higher than filled positions. In the middle eastern States salary levels of filled and vacant positions are equal and only in the Central and Far West do we find salary levels of vacant positions somewhat lower than those of filled positions. There does not appear to be a ready explanation for this, particularly in view of the fact that 20% of these positions pay maximum salaries ranging from $5,500 to $8,000. There appear to be a number of possible explanations for the fact that vacant sanitarian positions pay higher salaries than filled ones. They are as follows:

1. Lack of qualified applicants for higher level positions.
2. Unwillingness or inability to move to a new location.
3. Inadequate communication of information on vacancies to qualified potential applicants.
4. 7% vacant positions is a normal situation for the sanitarian group. There may be newly established or newly graded positions which are likely to pay more than existing positions.

Needed - College Graduates with No Experience

One-third of all the vacancies for sanitarians consist of positions for college graduates with no experience. These positions offered a median starting salary of $4,479. Among 675 filled positions with the same requirements the median starting salary was slightly lower—$4,374. A 1956 study of salaries paid to college graduates shows that graduates in the sciences at 20 schools were receiving salaries averaging $4,464.

Engineering graduates with no experience were in 1956 according to reports from over 100 engineering schools, receiving a median salary of $5,040 as compared with $4,800 in 1955. Officials at over 100 colleges and universities[9] estimate that 1957 starting salaries for engineering graduates will be in the $5,250 area. There were 15 positions for graduate engineers with no experience among the 371 departments in the study. Nine of these were filled with a median starting salary of $4,650, six were vacant with a median starting salary of $4,250.

Salaries, of course, are not the only requisites for effective recruiting. In a recent survey[5] sanitary engineering students indicated that among the

work conditions which they desired most were: "interesting work," "co-workers from whom one can learn," "doing independent work" and "making important decisions," in that order of preference. Of lesser importance were "salary," "job security" and "geographic area." It is not likely that science graduates would differ from engineers in this respect. There are other areas in which industry can more effectively compete for college graduates. Benefits such as paid trips for interviews, paid moving expenses, complete and free medical and hospital care programs, premiums for overtime work, annual bonuses, etc. make pay offers considerably higher than the actual salary scale of the job.

Of all the sanitarian positions in the study, 43 percent required no experience. This provides a broad base for the recruitment of inexperienced personnel, who can assume more responsible positions through effective career development. This was not the case among engineering positions where less than 10 percent required no experience; and these positions were limited to a small number of departments. More than one-third of these entrance positions were vacant probably because the salaries were extremely low. Such a situation is not likely to provide for an adequate supply of trained engineers for the more responsible positions in local health departments.

SUMMARY

A salary study of 371 local health departments representing 3,251 professional environmental sanitation positions was carried out by the Committee of Salaries of the Conference of Municipal Public Health Engineers. This is the first of five Sections of a report on this study prepared by the Committee.

Salary levels for directors of environmental sanitation programs and other professional categories are presented. Since 1951 the salary levels for such personnel have been rising at rates varying from 5% to 13% each year.

The salary levels generally paid to engineers in local health departments are far below those paid to engineers generally and engineers in county and municipal governments. As a result, the number of vacant engineering positions in local health departments continues to increase. There appears to be a serious shortage of entrance positions for engineering graduates with no experience.

The average top salary paid to a sanitarian in a sample of 35 local health departments has been rising more rapidly since 1954 than any other salary group studied. 6.8% of the sanitarian positions studied were vacant. 30% of these were positions available to college graduates with no experience.

There is considerable geographic variation of sanitarian and engineering salaries and of vacancies in sanitarian and engineering positions. With the exception of the Far West, there is an inverse relationship between the percent engineering positions vacant and the salary level of the vacant positions.

Subsequent sections of this report will discuss in some detail the relationship of experience, educational, supervisory, and program responsibility with salary levels among the positions included in the study.

Committee on Salaries

Walter A. Lyon, Chairman (J.M. ASCE) October 22, 1956
William T. Ballard

Herbert J. Dunsmore (A.M. ASCE)
Reinhart W. Koch
John W. Lemon
Eric W. Mood
Louis W. Pickles
Jack C. Rogers
Lester A. Sanger

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APPENDIX 1

Explanation of Abbreviations and Terms

Commissioner - The medical officer in charge of the health department.

Director of Environmental Sanitation - The person other than the commissioner responsible for the environmental sanitation program.

Lowest Salary - The lowest pay-step of the lowest salary in the department paid to a specified professional category (i.e. sanitarian, engineer).

Maximum Salary - The highest pay-step in the salary range of a position.

N.S.P.E. - National Society of Professional Engineers.

Present Salary - The salary paid to incumbent in May, 1956.

Professional Personnel - All sanitation personnel other than clerical or labor.

P.H.S. - Public Health Service.

Top Salary - The highest pay-step of the highest salary in the department paid to a specified professional category (i.e. sanitarian, engineer).

APPENDIX 2

States in Geographic Regions

New England
- Connecticut
- Maine
- Massachusetts
- New Hampshire
- Rhode Island
- Vermont

Middle East
- Delaware
- District of Columbia
- Maryland
- New Jersey
- New York
- Pennsylvania
- West Virginia

Southeast
- Alabama
- Arkansas
- Florida
- Georgia
- Kentucky
- Louisiana
- Mississippi
- North Carolina
- South Carolina
- Tennessee
- Virginia
- Puerto Rico
- Virgin Islands

Southwest
- Arizona
- New Mexico
- Oklahoma
- Texas

Central
- Illinois
- Indiana
- Iowa
- Michigan
- Minnesota
- Missouri
- Ohio
- Wisconsin

Northwest
- Colorado
- Idaho
- Kansas
- Montana
- Nebraska
- North Dakota
- South Dakota
- Utah
- Wyoming

Far West
- California
- Nevada
- Oregon
- Washington
- Hawaii

Form No. 1

Conference of Municipal Public Health Engineers
COMMITTEE ON SALARIES
1956 Salary Survey

(Please fill out one of these forms for each class of professional* positions in Environmental Sanitation Division)

__1-4. Name of Health Department:_____
State:_____

__5. Population served in 1956:_____
Title of Class of positions:_____

__6-7. No. of such positions vacant on May 1, 1956:_____

__8-9. No. of such positions filled on May 1, 1956:_____

__10. Is this the position held by the Director of Environmental Sanitation? (check one)
( ) yes ( ) no

__11. This class of positions falls into the following category: (check one)
( ) engineer, ( ) sanitarian or sanitary inspector, ( ) other professional* personnel (such as chemist, veterinarian, air pollution inspector, etc.) - indicate which:_____

Annual Salary

__12-14. Minimum:_____per year

__15-17. Maximum:_____per year

__18. Normal number of years from minimum to maximum:_____years.

Minimum requirements to qualify for class of positions

__19-20. What is the regular basic education required for appointment to this class of positions? (show by one check mark)
( ) less than high school;
( ) 1, ( ) 2, ( ) 3 years of high school; ( ) high school graduation
( ) 1, ( ) 2, ( ) 3, ( ) 4 years of college; ( ) bachelor's degree.
( ) 1, ( ) 2, ( ) 3, ( ) 4 years of graduate work; ( ) Master's degree.
( ) Ph.D. degree, ( ) other graduate degree:_____

__21. If a bachelor's degree is required under 19-20; check type required:
1 ( ) bachelor's in engineering
2 ( ) bachelor's in sanitary science or public health
3 ( ) any bachelor's in science
4 ( ) any bachelor's 5 ( ) other bachelors:_____

__22. If a master's degree is required under 19-20; check type required:
1 ( ) master's of public health, 2 ( ) master's of sanitary science
3 ( ) master's of sanitary engineering or public health engineering
4 ( ) other master's of engineering?_____(indicate type)
5 ( ) other master's of science?_____(indicate type)
6 ( ) other masters?_____(indicate type)
7 ( ) other graduate degree?_____(indicate type)

__CONTINUED ON PAGE 2__

* Professional positions include all positions in environmental sanitation except clerical and labor.

__23. Can experience be substituted for the education checked under 19-20? (check one) ( ) yes ( ) no

__24-25. If "yes" indicate by "x" mark under 19-20 the minimum education below which no substitution is accepted.

Amount and type of experience required to qualify for class of positions

__26. Minimum total number of years of experience required:_____yrs.

__27. Minimum number of years of experience required in general environmental sanitation:_____yrs.

__28. Minimum number of years of experience in a specialty in environmental sanitation required:_____yrs.

__29. Minimum number of years of supervisory or administrative experience required: _____ _____yrs.

__30. Is a professional license, or eligibility for one, required to qualify for the class of positions? (check one) ( ) yes ( ) no
Indicate type of license:_____

Responsibility held by class of positions

__31. Program responsibility of class of positions (check one)
( ) (1) Position class is a generalized one; it covers all or most programs handled by the Division of Environmental Sanitation.
( ) (2) Position class is a specialized one; it covers one or a few programs handled by the Division.

__32. Geographical Responsibility of class of positions (check one)
( ) (1) (A) Class of positions covers entire area covered by Department
(B) Class of positions covers only a portion of area covered by Department. (check most typical population range covered)
( ) (2) less than 25,000 ( ) (4) 50,000-99,000 ( ) (6) 200,000-499,000
( ) (3) 25,000-49,000 ( ) (5) 100,000-199,000 ( ) (7) over 500,000

Supervisory Responsibility held by class of positions

__33. Direct supervisory or assistant supervisory responsibility for the following average number of professional* personnel. (Include all professional personnel reporting through subordinate supervisors) (check one)
( ) none ( ) 5-9 ( ) 20-29 ( ) 40-49 ( ) 75 and over
( ) 1-4 ( ) 10-19 ( ) 30-39 ( ) 50-74

__34. If this class of positions is primarily of a staff, policy, consultant or training nature, indicate number of professional* personnel whose work assignments are affected thereby. (check one)
( ) none ( ) 5-9 ( ) 20-29 ( ) 40-49 ( ) 75 and over
( ) 1-4 ( ) 10-19 ( ) 30-39 ( ) 50-74

__35. Remarks:_____

_____

*Professional positions include all positions in environmental sanitation except clerical and labor.

Form No. 2

Conference of Public Health Engineers
COMMITTEE ON SALARIES
1956 Salary Survey

General Data

__1. Name of Health Department: _____

State: _____

__2. Does this Department provide direct local health services? ( ) yes ( ) no

__3. Population served in 1956: _____

__4. Present Salary of Director of Environmental Sanitation Division: _____ per year

__5. Present Salary of Commissioner of Health: _____ per year

__6. Number of Personnel in the Health Department other than Commissioner having a salary equal to or greater than Director of Environmental Sanitation: _____
List Title(s): _____

_____

__7. Do you wish to be sent a copy of the report resulting from this study? ( ) yes ( ) no

__8. Name of Director of Division of Environmental Sanitation: _____

_____

_____

Title _____

Office address _____
(City) (State)

__9. Remarks: _____

_____

_____

_____

_____

_____

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

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DISCUSSION
(Proc. Paper 1288)

Page

Biological Treatment of Highly Alkaline Textile Mill Waste-Sewage Mixture, Progress Report of the Industrial Waste Section of the Sanitary Engineering Research Committee. (Proc. Paper 750. Prior discussion: 822, 848. Discussion closed.) . . . . . ---*

The Mechanics of Reaeration in Natural Streams, by Donald J. O'Connor and William E. Dobbins. (Proc. Paper 1115. Prior discussion: none. Discussion closed.)
by Alex N. Diachishin 1288-3
by A. Pasveer 1288-10
by M. C. Rand 1288-12

Note: Paper 1288 is part of the copyrighted Journal of the Sanitary Engineering Division of the American Society of Civil Engineers, Vol. 83, SA 3, June, 1957.

* There will be no closure.

Discussion of
"THE MECHANICS OF REAERATION IN NATURAL STREAMS"

by Donald J. O'Connor and William E. Dobbins
(Proc. Paper 1115)

ALEX N. DIACHISHIN,* A.M. ASCE.—The authors are to be complimented for their analysis of reaeration and their attempt to relate reaeration coefficients with the hydraulic characteristics of a stream. The following comments are remarks of an elementary nature concerning the Dobbins-O'Connor formulations and present perhaps a fuller examination of some of the implications inherent in these formulations than were published in the original presentation. The discussion following has been ordered into three major categories:

A. Theory
B. Verifications of Theory by Field Data
C. Verifications of Theory by Experimental Data

Theory

Equation (35), defining one of the two classes of reaeration turbulence defined by the authors, can be amplified by the use of Chezy's formula. Solving Chezy's formula for the fourth root of s, we obtain

$$S^{1/4} = \frac{U^{1/2}}{C^{1/2} R^{1/4}} \tag{42}$$

where R is the hydraulic radius, the other notations remaining the same as in the original presentation. Further noting that for broad shallow streams the hydraulic radius is very nearly equal to the depth of flow, making this indicated substitution, as well as that shown in (42) in Equation (35), we obtain an alternate expression for non-isotropic turbulence

$$k_2 = \frac{g^{1/4}}{2.31 K^{1/2} C^{1/2}} \frac{D_L^{1/2} U^{1/2}}{H^{3/2}} \tag{43}$$

It is evident that this expression bears a striking resemblance to the authors formulation for the case of isotropic turbulence (37). A comparison of (37) and (43) indicates that the isotropic and non-isotropic expressions are equivalent when

$$g^{1/4} = \chi^{1/2} C^{1/2} \tag{44}$$

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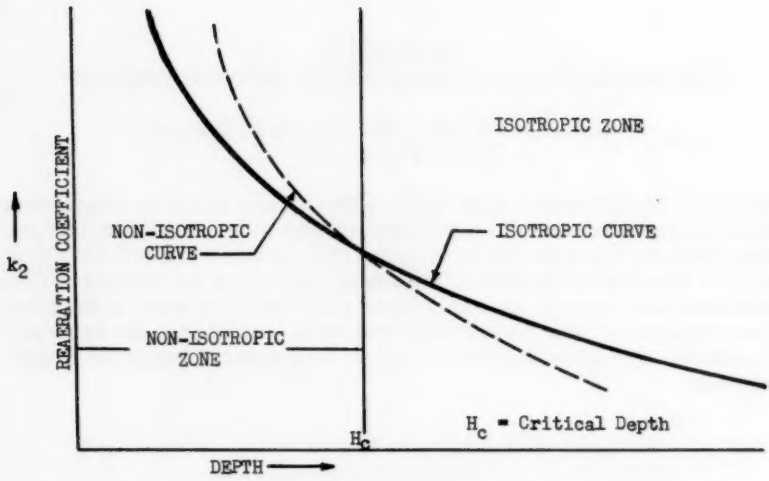

FIGURE 3. THEORETICAL EFFECT OF DEPTH ON REAERATION COEFFICIENT

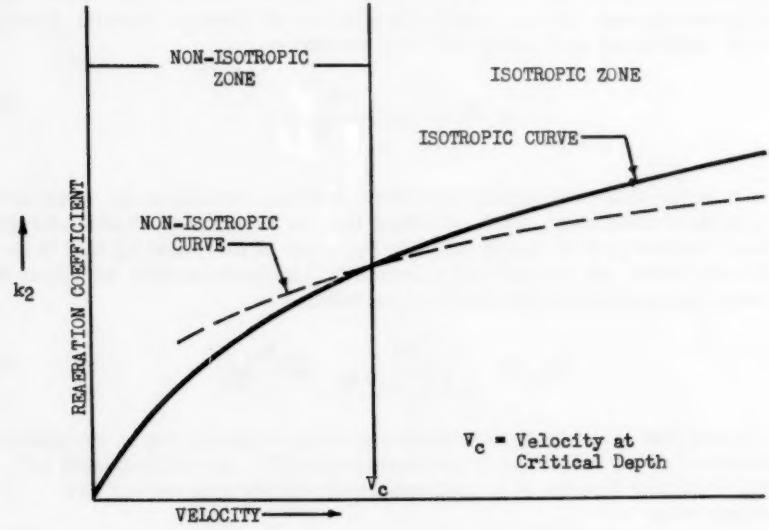

FIGURE 4. THEORETICAL EFFECT OF VELOCITY ON REAERATION COEFFICIENT

which gives the same value for the critical Chezy coefficient as does Messrs. Dobbins and O'Connor derivation. However, it is interesting and important to note that whereas the "constant" for the non-isotropic case,

$$\frac{g^{1/4}}{2.31 X^{1/2} C^{1/2}}$$

decreases with an increasing hydraulic radius, the isotropic constant, $\frac{1}{2.31}$ of course remains constant regardless of the value of the hydraulic radius.

The relation of k_2 and H, with constant velocity and temperature is shown in Figure 3. It is evident from the Figure that k_2 decreases with increasing depth for both the isotropic and non-isotropic zones, the only difference being in the "constants." Although Chezy's formula is not particularly applicable since the depth of flow in Chezy's formulation cannot change without a corresponding change in the velocity—slope being held constant—we can surmise that since the Chezy coefficient is proportional to the depth, the "constant" for the non-isotropic case tends to decrease with increasing depth. This decrease in the non-isotropic "constant" makes the k_2 values of the non-isotropic curve less than the corresponding isotropic values in the isotropic zone. However, regardless of the difference in the isotropic and non-isotropic curves, it is evident that with a constant velocity k_2 is expected to reach a maximum at low depths and to decrease continually with increasing depth.

The expected variation of k_2 with velocity—depth and temperature being held constant—is shown in Figure 4. From the Figure it is evident that both the isotropic and non-isotropic curves increase with increasing velocity and that the non-isotropic curve is above the isotropic curve in the non-isotropic zone and below the isotropic curve in the isotropic zone. If we use only those portions of the curves for the zones in which they are applicable, it is evident that an inflection point may occur at the velocity corresponding to the critical depth separating the two zones. Further it should be noted that the exponent of the velocity for the isotropic curve is $\frac{1}{2}$. Although the exponent of the velocity is also $\frac{1}{2}$ for the non-isotropic curve as shown by equation (43) it is possible that the value of this exponent would possibly be masked—due to the changing value of the "constant" in the non-isotropic case—if one were faced with the task of determining the value of the exponent from experimental data. However, it is clear that the exponential value for the velocity is equal to $\frac{1}{2}$ for velocities higher than the critical velocity, that is for velocities in the isotropic zone for the isotropic curve.

The allocation of isotropic and non-isotropic zones for both the examples shown in Figures 3 and 4 is somewhat arbitrary. Although the allocation of zones is quite clear for the constant velocity case, (Figure 3) it is not quite so clear for the constant depth case (Figure 4) and there is even some justification to reverse the allocation of zones. If this change were made—transposing the two zones and using only those portions of the curves for which they apply—it is apparent that the same inflection point would be maintained but that instead of k_2 taking an upward swing the curve would tend to flatten at the inflection point.

The conditions of either (A) constant depth and varying velocity or (B)

constant velocity and varying depth, do not occur except in laboratory studies. In stream flow the velocity is directly proportional to some fraction of the depth. We may, however, derive some useful knowledge concerning the variation of k_2 with both depth and velocity changing from an examination of equations (35) and (37). From (35), since a velocity term does not appear in the equation it is evident that k_2 decreases with increasing depth and that a maximum k_2 is expected at a minimum depth. In the isotropic zone, where (37) applies k_2 will decrease with increasing depths and velocities if

$$\Delta[U^{1/2}] < \Delta[H^{3/2}]$$

An examination of either the Chezy formula or others applicable to stream flow indicates that the inequality shown above usually holds, and hence we may conclude that increasing river stages and corresponding increasing velocities will cause decreasing k_2 values.

To summarize this examination of Messrs. Dobbins and O'Connor formulations:

A. Equations for both isotropic and non-isotropic turbulence are characterized by similar equations (37) and (43). For both cases k_2, the reaeration coefficient, is directly proportional to the $\frac{1}{2}$ power of the velocity and inversely proportional to the $\frac{3}{2}$ power of the depth.

B. With constant velocity and temperature k_2 decreases with increasing depth.

C. With constant depth and temperature k_2 increases with increasing velocity. The reaeration coefficient increases at an apparent rate no greater than the $\frac{1}{2}$ power of the velocity.

D. In streams maximum k_2 values will occur at minimum depths (and correspondingly low velocities). Minimum k_2 values will occur at maximum depths and correspondingly high velocities.

Verifications of Theory by Field Data

The use of equation (2) to determine reaeration coefficients is common as was noted by Messrs. Dobbins and O'Connor. All other terms being known, except for k_2, the solution for k_2 can be accomplished by trial and error by substituting in (2) and finally obtaining a value satisfying the equation, or by any of a variety of approximation methods. However, although the solution for k_2 is mathematically straightforward, the dependence of k_2 on the other factors in (2) is not to be minimized. These factors are at best only approximated and, therefore, give only approximate k_2 values. Perhaps the greatest sources of inaccuracy in reported k_2 values are the dissolved oxygen deficits. The change in the dissolved oxygen deficit from one point on a stream to a downstream point reflects not only the depletions due to the oxygen requirements of pollutional materials and the additions due to reaeration, but also the oxygen requirements of life forms not involved in the pollutional purification process. Recently, the discussor had occasion to compute reaeration coefficients utilizing oxygen deficits for both an eight hour and a twenty-four hour

period on successive days. There was a four fold difference in the computed coefficients. In view of these considerations it is all the more surprising to note the almost exact agreement between the coefficients calculated by Messrs. Dobbins and O'Connor and the observed reaeration coefficients.

In order to further test the Dobbins-O'Connor method a stretch of the Ohio River—Stations 598 to 611—from the same Ohio River survey data used by the authors was analyzed. (1) This data resulted from extensive surveys by the United States Public Health Service originally conducted in 1914 and 1915. The authors of the survey report classified Stations 598-611 as "relatively smooth," with an average low water slope of .316 feet per 1,000 feet. Chezy coefficients were computed and the type of turbulence assigned to each case, all in accordance with the procedure recommended by Messrs. Dobbins and O'Connor, even to the extent of applying the recommended empirical depth correction to be added to the river stage, equal to one half of the mean depth at extreme low water. Data for temperatures below 10°C were omitted as they were by Messrs. Dobbins and O'Connor. The computed results are shown in Table VI, and it is evident at a glance that there is a considerable discrepancy between the observed and computed results.

Another example, chosen at random, is for the Lower Little Tennessee River, the base data as given in a recent article.[38] The computed value indicates a k_2 of 1.28 per day, while the observed value was approximately 10.0. Still another example, in the discussor's recent experience, is for the Connecticut River which evidenced a computed value of 0.28 per day and an observed value of 0.56 per day.

It is not clear from these preceding examples whether the Dobbins-O'Connor method gives consistently low values or whether the observed deviations of the computed values from the field study values reflect variations from a mean value.

Rather more important than the agreement or lack of agreement of the observed with the computed values, is the variation of k_2 values at a given location with a change in depth and velocity. Sanitary engineers are often concerned with critical values of flow and reaeration rather than average values and the depth or flow at which minimum values of the reaeration coefficient occur may well be a matter of critical importance.

From the discussion in the previous section it is evident that according to the Dobbins-O'Connor theory, a minimum will occur at high river stages. An examination of the data in Tables IV and V of the original paper bears out this contention. Four out of five of the Ohio River stretches analyzed have minimum claculated k_2 values occuring at maximum river heights, while at the other station the minimum k_2 value occurs at an intermediate height. Contrast this occurrence of the computed minima with the observed k_2 minima. For one section the minimum occurs at maximum height, at two sections the minima occur at intermediate heights, and at the remaining two sections the minima occur at minimum heights, hardly a verification of the Dobbins-O'Connor theory.

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38. Churchill, M. A. "Effects of Storage Compoundments on Water Quality," Journal of the Sanitary Engineering Division American Society of Civil Engineers 83, No. SA1 (1957).

Verifications of Theory by Experimental Data

The authors state that "the critical point to be verified in the theoretical development is the relationship between the reaeration coefficient and the rate of surface renewal, as indicated by equation (28)." Equation (28) can be written in the equivalent form shown below when the depth is held constant as was the case in the experimental apparatus of the authors.

$$k_2 = G r^{1/2} \tag{45}$$

Where G is a constant. The test data presented presumably verifies the theory presented since it is reasoned that the speed of rotation of the experimental apparatus is directly proportional to the rate of surface renewal. The theory, according to equation (45) predicts that the reaeration coefficient varies as the square root of the rate of surface renewal and their proportionality is purportedly confirmed by the linear variation of the experimental data for k_2 with the square root of the speed.

Utilizing the criterion of the authors the authors experimental data was fitted by the method of least squares for both a linear and a square root function of the speed. From the results of the statistical analyses shown in Table VII it is to be noted that no significant change in the correlation coefficient occurs when a square root function is used instead of a linear function. It is unfortunate that the range of speeds employed in the experimental tests was so limited in range. With a larger range in speeds—extending perhaps several magnitudes—it would be possible to determine whether the effect of speed is more aptly described by a linear or a square root function. For the experimental range selected, all that can be said is that if indeed there is a square root effect its influence in the range of speeds selected is extremely minor.

Data from another experimental study[28] are also available to check the Dobbins-O'Connor formulations. The experimental data resulted from an extended series of flume experiments conducted by the United States Public Health Service. For a constant depth the results are as shown in Table VIII for velocities ranging from 1.6 to 4.6 feet per minute. As regards velocities greater than those for the range 1.6 to 4.6 feet per minute, the report presenting the results of the flume studies states:

> ". . . . . the plotted points are shown to follow two distinct straight line trends, the slope breaking sharply upward at a velocity of flow approximating 10 feet per minute. If the slope of each line based on the dearated water tests be designated by (n) in the function (V^n), where (V) is the velocity of flow, the numerical value of (n) for the steeper line approximates 1.75 and the flatter line 0.50."

Thus, it appears that at best the authors theory is applicable to low velocities below 10 feet per minute, and certainly not for the entire range of velocities.

As regards the effect of depth, the same report[28] states:

> ". . . . . Little if any indication was evident from the foregoing data as to any consistent influence exerted by variations in the stream depth on the rate of surface reaeration, though plots of the data. . . . . have shown a tendency towards slightly increased surface rates of reaeration with increased depths. It is possible in the case at hand the differences in flow depth were too small to bring out clearly any well-marked effect of this

TABLE VI

REAERATION COEFFICIENTS - OHIO RIVER (1)
STATIONS 598-611

Mean Depth Feet	k_2 - computed per day	k_2 - observed per day	Remarks
27.2	0.067	0.170	Isotropic
12.7	0.127	0.613	Non-isotropic
11.8	0.144	0.925	"
10.9	0.156	0.792	"
11.3	0.143	1.140	"
11.3	0.135	0.446	"
Average	0.129	0.514	

TABLE VII

CORRELATION COEFFICIENTS FOR EXPERIMENTAL DATA

	Deaerated Water	Sulfite Solution
Linear	0.994	0.993
Square Roote	0.995	0.991

TABLE VIII

EXPERIMENTAL REAERATION COEFFICIENTS (28)

Temperature °C	k_2 per day
0-10	$0.32V^{0.43}$
10-20	$0.50V^{0.50}$
20-30	$0.75V^{0.51}$

variable, except to suggest that where the depth was increased, with a given velocity of flow, some hydraulic condition may have caused a slightly greater degree of turbulence than with more shallow depths."

Although the preceding statement referred primarily to shallow depths, the conclusion of the studies considering all depths was essentially the same, expressing the opinion that the variation of the depth of flow appeared to exert a slight influence. These conclusions do not lend evidence to the authors theory which predicts a considerable effect of height on the reaeration coefficient.

CONCLUSIONS

There is a substantial area of disagreement between some of the results forecast by the Dobbins-O'Connor theory and those observed in other laboratory and field studies. Inasmuch as it is difficult to have prior knowledge of those instances for which the authors' theory will apply it would appear that actual measurements of reaeration coefficients still constitute good practice.

A. PASVEER.[1]—The work on the mechanism of reaeration in natural streams by Messrs. O'Connor and Dobbins proves that it is important to know that the turbulent flow of a river follows certain laws, resulting in a rate of surface renewal. This rate can be calculated on a theoretical basis and is one of the factors which determines the rate of reoxygenation of the river water.

This study doubtless is not only of great value for an understanding of the reaeration process but also for its use in practice, i.e. for the evaluation of the natural purification capacity of a stream.

It must be stressed, before proceeding further, that the following remarks are not meant to decrease the value of the work done by Messrs. O'Connor and Dobbins in any way.

The writer objects to the use of the Lewis and Whitman two-film theory in theoretical considerations of the reaeration of water. Though the two-film theory has been of great value for practical work it is the opinion of the writer that the use of an overall coefficient of diffusion for a gas film (K^g) or a liquid film (K^L) obscures what is really going on in the layers at the interface between the air and water.

Especially with regard to the oxygenation of water there is no need for assuming a liquid film; indeed, it can be shown that, most probably, in the aeration with air bubbles there is no (more or less) "stagnant" liquid film at all.(1)

In the process of dissolving a gas into a liquid, the "resistance" in the boundary layer is not caused by the existence of a "stagnant film." The "resistance" is due to the fact that the rate of dissolving of a gas in a quiescent liquid is inversely proportional to the square root of the time. The rate of dissolving is a maximum at the moment of creation of the surface and then rapidly decreases with time.

The writer's principal objection to the two-film theory is that this theory with its formula, as it is used in the aeration of water,

$$Q_d = AK_L (c_s - c_L) t \text{ , } (K_L = \text{overall liquid film coefficient})$$

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is not always restricted to practical purposes in nonquiescent conditions and that it obscures the fact that according to the theory of diffusing the quantity diffused is proportional to the square root of the time and not to the time. This may lead to erroneous conclusions. For instance Haney[2] says that "For a given situation as long as the product of $\frac{A}{V}$ and t remain constant, the performance will be the same." This may hold true in most practical situations, however, it certainly may not be generalized as a theoretical reasoning.

The exact physical formula deduced by Stefan[3, 4] from Fick's law of diffusion in the case of a poor soluble gas into quiescent water is:

$$Q_d = 2\,A\,(c_s - c_L)\sqrt{\frac{kt}{\pi}}$$

k being the coefficient of diffusion of oxygen in water (sq cm /sec).

From the two formulae given it follows that for quiescent water $K_L = 2\sqrt{\frac{kt}{\pi}}$, from which there clearly follows that in this case the value of K_L depends on the time of exposure.

In most practical conditions of nonquiescent water the formula of Lewis and Whitman will hold true because, due to the rate of surface renewal, there results an "overall coefficient" constant for each unit of time. However, from this we cannot learn what exactly is going on at the interface.

The coefficient of diffusion k is a physical constant, the numerical value of which constant is equal to the quantity of matter, which diffused in unit of time through the unit of area, if at both sides of the surface there is the unity of concentration gradient.

In the case of the diffusion through an interface between air and water there is a concentration gradient only at one side of the interface. As Stefan has deduced, in this case the general formula is not:

$$Q_d = A\,(c_s - c_L)\sqrt{\frac{kt}{\pi}} \quad \text{but} = 2\,A\,(c_s - c_L)\sqrt{\frac{kt}{\pi}}$$

So in the work under discussion equation (11), for instance, should be

$$N = 2\,(c_s - c_L)\,(\frac{D_L\,t}{\pi})^{\frac{1}{2}}.$$

The authors express some doubt as to the application of this formula to a liquid film of small thickness. Stefan made a detailed investigation of the conditions under which the formulas evolved by him are applicable. His conclusion was that the equation shows no greater divergence than 1% as long as the condition is fulfilled that $\frac{\pi^2 kt}{h^2} < 3$ (h = column of water) and no greater divergence than 2% as long as the value of this expression is less than 4.

From this it may be calculated, for instance, that for a diffusion time of 1 sec the minimum column length at which the divergence is less than 2% is 60 micron.[5]

From the foregoing it may be concluded that the writer's preference might have gone to a theoretical reasoning, on the base of the use of Stefan's formula

$$Q_d = 2 A (c_s - c_L)\sqrt{\frac{k}{\pi}} .$$

However, as said before, this is insignificant as to the results Mr. O'Connor and Mr. Dobbins have obtained and to the progress that is made in our knowledge on river reaeration.

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M. C. RAND.[1]—This paper represents an important advance, both in our theoretical understanding of the mechanism of the reaeration process, and in the field application of that understanding. Several points stand out as being of particular interest:

1. The careful and nearly rigorous derivation of the reaeration constant (k_2) as a function of the rate of surface renewal. While expressions of the same, or nearly the same, form have been previously presented, the present derivation sets forth the range of applicability of the final equation more clearly than most previous reports.

2. The expression of the rate of surface renewal, and hence of k_2, in terms of parameters susceptible to independent measurement. Thus, for the first time, it becomes possible to estimate reaeration in a stream by methods which do not require prior observation of the reaeration rate.

3. The application of the modern theory of turbulence to reaeration, in cases involving a flow of finite velocity and definite direction, suggests a possible method of attack for the problem of reaeration in other bodies of water, such as lakes and reservoirs, in which turbulence is due primarily not to flow, but to wind action, thermal convection, etc.

In deriving the general equation for reaeration as a function of surface renewal, the authors employ the assumption that the surface layer of infinitesimal thickness is saturated with dissolved oxygen instantaneously upon the exposure of a new element of surface. This assumption imposes a slight limitation upon the generality of the resulting equation, since the saturation of the surface layer must require a length of time which is finite, however small. The limitation does not become important, however, so long as the average time of exposure of an element of surface is long compared with the time required for saturation of the surface layer. In other words, the assumption excludes only the state of extreme turbulence which has been termed a "heaving surface."

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The equations presented apply, and are intended to apply, only to bodies of water in which there exists a flow of finite velocity and measurable direction. The problem of reaeration in relatively quiet water, exhibiting no net flow, remains unsolved. However, this work may very well point the way for an extension of the same reasoning to cases of this type.

DIVISION ACTIVITIES
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

June, 1957

TWO HUNDRED FIFTY NINE ARE CERTIFIED

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ILLINOIS WATER AND DROUTH STUDY COMMISSION REPORTS TO LEGISLATURE

On February 15, 1957, the Illinois Water and Drouth Study Commission submitted its report to Governor William G. Stratton and the 70th General Assembly of Illinois. A 1955 Act of the Illinois General Assembly created the Commission to study the water and drouth situation in Illinois, to determine surface and underground water rights, and to recommend legislation.

The Commission reported that there was sufficient water in Illinois to supply all present needs but, that although there is an abundance of water in the rivers and underground supplies of the State and in Lake Michigan, accessibility and distribution constitute a major problem.

They also stated that existing State technical, advisory and enforcement agencies operating in the water resources field are functioning effectively and should be continued and strengthened and that there was a need for a more comprehensive study report on Illinois water resources than the Commission had been able to make.

The Commission recommended that the primary resources policy of the State be concerned with development and conservation of supplies in each area, guided by knowledge and investigation, rather than appropriation and priority.

They also recommend a strengthening of the existing Water Resources and Flood Control Board by provision of a full-time executive secretary and a professional and clerical staff to coordinate water resources activities and that the board prepare a comprehensive study report on Illinois water resources.

PURDUE UNIVERSITY UNDERTAKES FARM POND WATER TREATMENT STUDY

In cooperation with the Indiana State Board of Health, the Purdue University Sanitary Engineering Department and the Agriculture Engineering School have undertaken a study of farm pond water supplies to determine the effect of watershed land management practices on pond water quality; determine which

water pre-treatment practices are applicable to a farm water filter system; and develop or adapt a water treatment facility that is capable of producing potable water by treating farm pond water.

The initial phase of this study was started in February, 1957, with funds appropriated by the Purdue Experiment Station. These funds are to provide a water supply for a staff home at Purdue's Southern Indiana Forage Farm by whatever method shows the most promise. A slow sand type of treatment unit is being installed that will utilize water from a nearby farm pond. Farm pond water demands for both domestic and farm use will be studied to establish pond and watershed requirements.

An application is also being made to the Public Health Service for a research grant to study the land management of the watershed as it affects pond water quality. Of particular concern is the effect of fertilizers used on the watershed on the nitrate concentration in the pond water, and likewise, how phosphates, lime, insecticides, etc., affect the pond water quality.

Pre-treatment of the pond water within the pond anticipates a study of the use of gypsum, inexpensive in southern Indiana, as a clarifying agent. Another phase of the project to be studied is the use of roughing filters over the intake pipe in order to reduce the turbidity that would otherwise have to be removed by the filter plant.

The overall research project anticipates a comparative study of various treatment methods, including adaptations of the slow sand filter and applications and modifications of commercial-type diatomite filters. Comparative studies will be made of the diatomite vs slow sand filters with particular concern of their adaptability to the farm operations. Numerous ponds are available with varying watershed characteristics.

This study recognizes the desperate need for a simplified surface water treatment plant that is capable of being employed in the rural home. Much of southern Indiana and many other states have areas that lack adequate ground water, but can be provided with farm ponds. The Soil Conservation Service has provided technical assistance in southern Indiana for farm pond construction for livestock use at a rate of over 1000 per year in the past several years. Recent drouth in this area has accentuated water deficiency problems and has resulted in hauling water large distances for livestock and household use. The farm pond might be utilized as a safe water source, but at the present time there is no definite recommendation to be made to these farm people as to how farm pond water is to be satisfactorily treated. It is hoped that this research will outline step-by-step procedures for the farmer to follow in the construction of his farm pond water treatment facility.

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A New Graduate Program in Sanitary Engineering

The Illinois Institute of Technology, has announced a program of graduate study in sanitary engineering which, both in structure and emphasis, is a departure from the traditional offering in the field. The program has as its prime objective the educational preparation of an engineer-scientist who is capable of performing with equal facility in any of the many branches of sanitary engineering.

The program is a result of a careful study made of the educational requirements of contemporary and future sanitary engineering technology. These

requirements were found to be so diverse that, at the Master of Science level, that it was decided not to force the student to specialize so early in his career, but to seek a program of study that would permit him to cover the entire field of sanitary engineering, perhaps not in detail but at least in the fundamentals.

Four courses constitute the core of the program.

Sanitary Science - A lecture course covering those principles of inorganic chemistry, organic chemistry, biochemistry, nuclear physics, and biological decomposition which are basic to sanitary engineering analyses.

Sanitary Engineering Analysis - A lecture and laboratory course emphasizing the derivation and analysis of data for use in rational design.
Prerequisite: Sanitary Science

Unit Operations and Processes in Sanitary Engineering - A lecture course covering the rational design of the unit operations and unit processes employed in sanitary engineering.

Design of Sanitary Engineering Treatment Processes - A lecture and design laboratory course dealing with the application of unit operations and processes to the design of the over-all treatment processes.
Prerequisite: Unit Operations and Processes

Regardless of the advantages offered by employing a structural rather than a functional treatment of subject material, the success of such a program will depend primarily upon where the emphasis is placed. Consequently, every opportunity will be taken to stress principles rather than practice. It is believed that with this emphasis the above curriculum will produce a creative and versatile sanitary engineer.

The program will be available both in the day and evening divisions. Details may be obtained by writing to the Department of Civil Engineering, Illinois Institute of Technology, Chicago 16, Illinois.

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DID YOU KNOW THAT —

The Robert A. Taft Sanitary Engineering Center of the Public Health Service, Cincinnati, Ohio, recently participated in the 100th anniversary exposition of the Cincinnati Academy of Medicine with an exhibit depicting its four fields of research—air pollution and control, water supply and pollution control, radiological health, and milk and food sanitation. The exhibit was later displayed at the 42nd Industrial Health Conference in St. Louis and is scheduled for display in Washington, D. C.

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A 112 page report "Water Quality and Flow Variations in the Ohio River" is available from the headquarters of the Ohio River Valley Sanitation Commission, Cincinnati 2, Ohio. The report is a record of four years (1951 - 1955) of continuous chemical and bacteriological analyses from a network of Ohio River monitor stations. The publication includes a hydrographic study of the flow variability pattern of the River, particularly minimum flows. Cost of the publication is $2.00.

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DID YOU KNOW THAT —

An editor cannot prepare an interesting issue unless the members will submit interesting news items—And this month you did not do your part. If you like this section of the journal you must take the time to write and mail the material to the Editor not later than the 15th of odd months. If you don't care, don't write anything and we will all relax and quit trying. Please cooperate by sending news items of interest to: Paul W. Reed, Editor, Division Affairs Section, SED Journal, Division of Sanitary Engineering Services, Public Health Service, Department of Health, Education and Welfare, Washington 25, C. D.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

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JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998 (SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006 (HY3), 1007(HY3), 1008(HY3), 1009(HY3), 1010(HY3)[c], 1011(PO3)[c], 1012(SA3), 1013(SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c]; 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

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FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW1), 1245(HW1), 1246 (HW1), 1247(HW1), 1248(WW2), 1249(HW1), 1250(HW1), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW1), 1257(IR1)[c], 1258(HW1)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

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c. Discussion of several papers, grouped by Divisions.

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